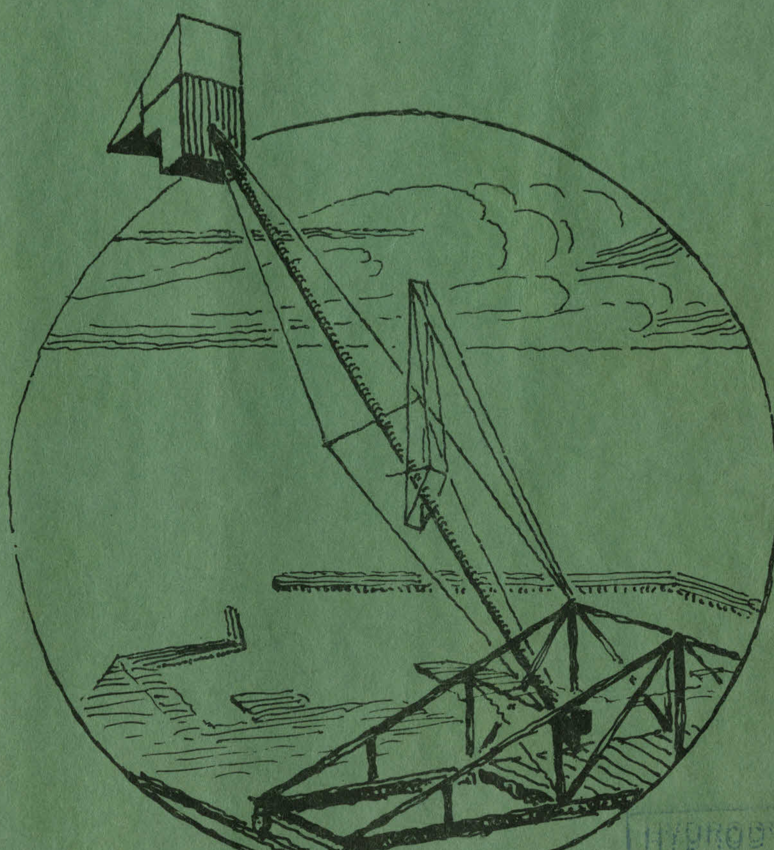
WAVE AND SURGE STUDY
FOR THE
NAVAL OPERATING BASE
TERMINAL ISLAND, CALIFORNIA



HYDRAULIC STRUCTURES LABORATORY
THE CALIFORNIA INSTITUTE OF TECHNOLOGY
PASADENA
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SUMMARY

OF

CONCLUSIONS AND RECOMMENDATIONS

This report presents the results of the model study of the water movements induced by waves and surges in the area adjacent to the Naval Operating Base at Terminal Island, California. The objective of the study was to test the designs proposed by the Navy for the construction of a mole to enclose this critical area, and to recommend such changes in these designs as are necessary in order to secure the desired protection fron these water movements.

The following conclusions and recommendations have been selected from the complete tabulations contained in Section VIII, Pages 197 to 204. The numbering corresponds to that employed in Section VIII.

A. CONCLUSIONS

(4) Field observations show that wave trains with periods varying from about 10 seconds to one hour occur in the drydock area. Fifteen second and three minute periods are common. Both field and model results show that the three minute surge waves are responsible for major ship movements and damage. More specifically, surges with periods of from one to three minutes which have amplitudes of two-tenths of a foot or greater will induce ship motion capable of causing appreciable damage. On the other hand, the fact that the drydock gates were handled during such surge conditions without difficulty indicates that surge motion does not interfere seriously with this phase of drydock operation.

(5) Very little, if any, damage occurs to ships and piers during times of abnormally high waves which have periods of about fifteen seconds. Conversely, the evidence indicates that these fifteen second wave trains do interfere with the operation of the drydocks and cause damage to the gates and seats.

(8) A properly designed mole will give adequate protection to the entire inner basin from the effects of 15 second wave trains and less complete protection from one minute and longer surge trains.

(9) Neither of the designs proposed by the Navy, namely, the "Original Mole" or the "Standard Mole" is capable of affording the required degree of protection to the inner basin without significant modifications. The major change required is the reduction of the gate opening from 2070 ft. to 750 ft. However, appreciable improvements also result from modifying the

mole aligment and altering the exact location and shape of the gate opening.

(10) The reduction of the gate opening in the mole is particularly necessary to reduce the amount of surge-produced movement in the inner basin. The entire mole and Pier A extensions at the gate must be of tight, or opaque, construction to be impervious to the effects of one minute and longer period surges.

(11) The mole with the 2070 ft. gate opening affords considerably more protection from the short period waves than from the one minute and longer surges. However, even for the short period waves, the reduction of the opening to 750 ft. results in a significant improvement of conditions within the basin.

(17) Uniform conditions do not prevail over the entire basin. Vertical amplitude maps indicate locations of "live" and quiet areas. Typical live areas are found in the northwest corner of the basin, along the inner side of the mole near the gate, and also along the south side of Pier E extension.

(22) The effective fundamental basin resonance period is approximately six minutes. Wave trains of this period produce disturbances having the greatest magnifications. The motion within the basin with this train has an amplitude as large, and in some areas slightly larger, with the mole in place than without it. The occasional occurrence of surges of this six minute period must be anticipated.

(25) Although many valuable field measurements have been made, the total amount of reliable information concerning the overall conditions in the harbor area is still completely inadequate to serve as a basis for technically sound developments. The mole under consideration is a typical example. To a considerable extent, the conditions to be investigated in the model study had to be established on the basis of experience and judgment, rather than on factual data obtained from quantitative measurements. Therefore, in order to insure the reliability of the results, a much more extensive model study has been required than would have been the case if the field conditions had been known more precisely. Adequate field measurements are difficult to obtain. The existence of a series of standing wave patterns for the different wave trains introduces a serious complication. Due to this situation, records from a single station are of limited value. Such a record gives the summation of all the factors producing the disturbance at that particular location, but does not furnish sufficient information to permit the

breaking down of the disturbance into its component parts, including their amplitudes, periods and directions of travel. To do this requires a set of carefully selected synchronized stations. All of the qualitative evidence indicates that surges with periods of the order of three minutes cause the major portion of ship and pier damage. However, quantitative measurements of their vertical amplitudes are very fragmentary, and of their horizontal amplitudes and velocities are nonexistent.

B. RECOMMENDATIONS

(1) It is recommended that the alignment of the West or main mole be made in accordance with Figure 137; i.e., with two arms at right angles, one perpendicular and one parallel to the shoreline, connected with a diagonal arm inclined at 30° to the parallel arm and 3,000 ft. long.

(2) It is recommended that the gate opening be restricted to 750 ft. width, toe to toe of slope. This opening should be centered on the existing 45 ft. deep channel from the east breakwater gate to the drydocks area. The plane of the opening should be perpendicular to the channel.

(5) It is recommended that, in the planning of any additional construction within the basin, due consideration be given to the existing disturbance pattern:

(a) To determine the suitability of the site from the standpoint of the local disturbances and their effect on the proposed activity.

(b) To estimate the effect of the proposed construction on the disturbance pattern in the vicinity of other critical areas.

Such information is presented in this report, Paragraph (e), Pages 101 to 108, for specific designs and locations of new drydocks, marginal wharf and piers, etc., which have been proposed by the Naval Operating Base.

(9) It is recommended that a continuing program of field measurements and studies be initiated under competent technical research direction, independent of the operational activities of the Base, and that the scope of such studies should include:

(a) Determination of amplitudes, periods and direction of travel of the various existing wave trains.

(b) Determination of the standing wave pattern produced by these trains.

(c) Measurements of the horizontal amplitudes and velocities of the surge currents, and correlation of this information with the corresponding vertical amplitudes.

(d) Study and measurement of ship motions under normal, surge, and storm conditions, and correlation of these motions with the horizontal and vertical water movements.

ACKNOWLEDGEMENTS

This study was initiated by Captain J. J. Gromfine, Officer in Charge of Construction, Naval Operating Base, and carried on under his auspices until the time of his transfer. The study has been completed under the present Officer in Charge of Construction, Captain H. E. Wilson. The original arrangements were developed in detail with the assistance of Lieutenant Commander Gordon A. MacDonald, Resident Officer in Charge of Construction, who maintained close liaison with the progress of the work from its inception until he left for overseas training in June, 1944. Commander MacDonald made many valuable contributions to this work in the delineation of the problem, in the collection of pertinent field data, and in making the necessary arrangements for personnel and material to carry on the study under the existing emergency conditions. During the latter part of the study, his functions have been carried on by Lieutenant Commander A. S. Porter, his successor, who has been likewise very helpful.

Mr. Frank F. Mead, Civilian Engineer in charge of Land Acquisition and Development of the Naval Operating Base, has rendered invaluable assistance in collecting and evaluating field data, particularly in making available his long and intimate experience with the harbor area.

The study was authorized and sponsored by the Bureau of Yards and Docks, Navy Department, Washington, D. C., under the immediate direction of Captain W. L. Richards. Close liaison was maintained with his office through Mr. Nelson M. Brown, Project Manager, Bureau of Yards and Docks. Mr. Brown has contributed much to the study through personal consultations, both in Pasadena and Washington.

During the course of the study there has been a free and helpful exchange of ideas with the U. S. Waterways Experiment Station, Vicksburg, Mississippi. This laboratory has been carrying on a study which has closely paralleled the one in Pasadena. Captain Joseph B. Tiffany, Jr., Corps of Engineers, Executive Assistant, Mr. Fred R. Brown, Engineer, and Mr. B. Y. Hudson, Engineer, have each visited the Hydraulic Structures Laboratory at Pasadena and have all contributed to the progress of the study.

Under the existing conditions, this study could not have been made without the wholehearted cooperation of Mr. Arthur G. Randall, Civilian Engineer, and his assistant, Mr. E. G. Callard, who were assigned to the laboratory to supervise, under the direct-

ion of the laboratory staff, the actual construction and operation of the model and the calculation of the data.

A large part of the data was recorded photographically. Much of the success of this phase of the work was due to the initiative, ingenuity, patience, and skill of Mr. Edison R. Hoge of the Scientific Staff of the Mount Wilson Observatory who undertook this photographic responsibility in addition to his regular duties at the Observatory.

Valuable contributions to the physical nature and the mathematical expression of the phenomena underlying this problem were made by Dr. Harry Bateman, Professor of Mathematics, Theoretical Physics, and Aeronautics, at the California Institute of Technology.

At the time the Laboratory was requested by the Naval Operating Base to undertake this study, it was felt that under the existing conditions it would be impossible to do so unless the assistance of the staff and the use of the facilities of the Cooperative Laboratory of the Soil Conservation Service and the California Institute of Technology could be obtained. Consequently, authority for this assistance was requested by the Bureau of Yards and Docks from Dr. M. L. Nichols, Chief of Research, Soil Conservation Service, Washington, D. C., and Dr. Hugh H. Bennett, Chief of the Soil Conservation Service. The fact that this authority was immediately forthcoming made this study possible. Under this arrangement, Dr. Vito A. Vanoni, Research Project Supervisor of the Soil Conservation Laboratory, has acted as Associate Director of all phases of the study and Dr. H. A. Einstein has participated actively in it through the construction, operation, and analysis of the results of the ripple tank pilot studies, as well as through other contributions to the analytical background of the model work. He has assisted also in the preparation of the section of this report devoted to the ripple tank.

I. STATEMENT OF PROBLEM

This study has been made for the purpose of investigating the effectiveness of a proposed mole, designed to protect the Drydock Area at the Naval Operating Base, Terminal Island, California. The scope of this investigation was intended to include evaluation of the various parts of the design and the recommendation of alternate features if the study proved them to be more advantageous.

The reason for the construction of the mole is to eliminate objectionable conditions of water movement which have been interfering with the docking of ships, maintenance of mooring lines, and operation of the drydock gates and facilities. This water movement also has been causing ship movement of sufficient magnitude to result in damage to fender pile systems and piers. This objectionable water movement is the result of combination of waves and surges. This study has investigated the effectiveness of the various features of the mole configuration and allied constructions as means for reducing the water motion within the basin. On the other hand, this study has had no connection with the design of the mole section or the method of construction.

This study was authorized by the Bureau of Yards and Docks through a letter of intent of December 24, 1943, from the Officer in Charge of Construction, Naval Operating Base, Terminal Island. The terms of the study were based on a memorandum prepared by the Hydraulic Structures Laboratory of the California Institute of Technology. This memorandum was in response to a request from the Officer in Charge of Construction, who wished to ascertain whether or not the Institute would be willing to carry out such a study and, if so, under what conditions. In this memorandum the following items were stressed:

(1) Very little reliable information was available at the beginning of the study concerning the actual nature of the water motion which produced the damage to ships, piers, and drydock gates and gate seats.

(2) Although it was contemplated that considerable effort would be expended to obtain such measurements, it was recognized that conditions capable of causing severe damage occurred at irregular intervals and that there was no surety that the required information could be obtained before the study was completed.

(3) That the urgency of the study was such that it would be

impossible to devote adequate time to it to cover the range of possibilities that would be investigated under normal circumstances.

(4) That if the model study were undertaken, the inherent limitations put on it by the above conditions would have to be recognized.

II FIELD OBSERVATIONS OF NATURE AND SCOPE OF PROBLEM

A. FIELD DATA REQUIRED

In general, a model study of any kind is made to determine conditions at specific points of interest resulting from certain general conditions obtaining outside the problem area. In order to know how to operate the model and represent conditions in nature, it is necessary to know the conditions at the boundaries of the area covered by the model. For instance, in a river study it is necessary to know the size of the flood and the shape of the flood hydrograph at the upstream boundary of the reach which is being studied in the model in order to study conditions during flood stages. Similarly, in any model study involving wave action, it is necessary to know, at some boundary, what are the waves coming into the problem area from the seaward side. For the purpose of the study at the Naval Operating Base it was necessary to know the characteristic periods, heights, and direction of approach of the wave trains coming into the harbor area during times of excessive motion when difficulty was experienced in berthing ships, and during normal times, i.e., when ships could be handled without difficulty.

B. INFORMATION FROM EYE WITNESSES

The first step in collecting data necessary for the model study was to interview personnel who were familiar with conditions in the drydock area and in the Los Angeles Harbor in general. Personnel with whom these problems were discussed included a number of civilian harbor engineers and Naval architects and several Naval officers with extensive experience in handling ships within the harbor area. A number of conflicting opinions were encountered regarding just when conditions of unusual ship motion in the drydock area and in the harbor actually took place, and even more difficulties occurred in the evaluation of the relative intensity of the motion. This indicated that a given wave action or surge condition produced varying degrees of interference with operations of different kinds. It also indicated that during a given time the action at different points in the harbor was probably different. The various individuals were in agreement that ships ran on their lines, indicating that there were some periodic currents or surges of water which moved the ships. However, there was very little information on the actual cause of these movements or surges. A number of individuals attributed the surge to seiches set up in the harbor, but there was no reasonable explanation of how such seiches were excited. The explanations advanced included differences in atmospheric pressure over the harbor area and seismic disturbances. In this connection it is well to remark that for the purpose of this study it is not necessary to know the exact cause of the motion as long as the motion itself is known. However, the source of the motion is of considerable interest because once it is known, a more complete analysis of the problem is possible and the whole study is placed on a sounder basis.

One important question that had to be answered before the model study could proceed was, "How much protection did the outer breakwater afford to the harbor area?" Another way of stating this question is, "How much of the wave action, if any, could come through the breakwater?" This is an important question, since it will determine whether all of the disturbances come in from the navigation entrances and the open east end of the harbor, or whether disturbances can come in all along the breakwater. There seemed to be considerable difference of opinion on this question, thus indicating that further investigation was necessary to obtain the information needed to settle this point.

C. VISIBLE EFFECTS

Among the most important observations were those made visually and qualitatively during visits to the problem area. It was always possible to see waves in the drydock area with periods of from 10 to 20 seconds and with heights varying from 1 foot to 5 feet. Ship motions varied through a rather wide range and through a number of different characteristic motion patterns. At times the ships would lie at their berths with slack lines and with very little motion. At other times there was a very marked oscillatory motion involving longitudinal and transverse movements of 10 feet or more. It was during these rather large oscillatory motions that mooring lines were broken, ships and piers were damaged and the work on the ships was interrupted. In the course of observing these actions, it was noted that the periods of oscillation were rather long compared to the periods of the visible waves. The periods of the waves were measured in seconds, whereas those of the ship oscillations were measured in minutes and varied within the range of, say, 1 to 3 minutes. This important elementary observation indicated that the damaging ship motion could not be attributed to the visible waves and that some other type of disturbance existed. This tentative conclusion was also supported by other observations of paradoxical nature. It was observed that damaging ship motion did not necessarily occur during times when the drydock area was visibly rough, i.e., when the visible waves were unusually high. On the contrary, some of the worst damage to ships and docks was sustained during a condition of apparent calm.

Another operation that was interfered with by the motion of the water was the handling of the drydock gates. Considerable damage occurred to the seats of the gates on several occasions. The oscillating motion of the gate appeared to be of a relatively shorter period than the damaging motion of the ships. Also, it appeared that the gates could be handled without difficulty during periods of apparent calm but when ship motions were excessive. These observations indicated that the shorter period waves, i.e., the visible waves, were probably causing the damaging motion to the gates. The term "visible waves" is a relative one. A wave having an amplitude of 2 feet and a wave length of 600 ft. would probably be visible to careful observation if the water surface were smooth and free from surface chop. On the other hand, a wave of the same 2 foot amplitude but with a 6,000 ft. wave length would not be visible to the eye. The 600 ft. waves are common in the basin. A 6,000 ft. wave of 2 ft. amplitude would be a tremendous surge of about a 3 minute period.

The visual observations of ship motion, wave motion, and drydock gate behaviour, formed a basis for further detailed observation and measurements of wave motions and disturbances in the drydock and harbor area and, as the evidence presented below will show, served to clarify the problem.

D. PREVIOUS MEASUREMENTS

Tide gages have been maintained in the Los Angeles and Long Beach harbor areas by the United States Coast and Geodetic Survey (USC&GS) and other agencies for an extended period of time and considerable valuable information has been collected regarding tide and wave action. Some of these data have been published (1, 2) in papers which analyzed harbor conditions. A very valuable and comprehensive study of the entire harbor area was made by the United States Coast and Geodetic Survey (USC&GS) in 1935 and 1936. (1). A summary of the findings of this study is shown in Figure 1, which is a chart covering the Los Angeles and Long Beach harbor areas showing the observation stations. The stations are indicated as dots and the data observed at the station are summarized in notes adjacent to the dot. Two types of stations known as tide stations and current stations were used in this study. In the tide stations, which are designated on the chart by letters, conventional float type water level recorders were maintained. At current stations, which are designated by numbers, observations of current were made from a barge by means of Price and Eckman current meters and by timing floats. The current data from float measurements are considered more reliable than those from current meter measurements.

In this Coast and Geodetic Survey report the terms seiche and surge are used to denote, respectively, the surface disturbance indicated by a wave and the periodic horizontal flow of the water. Therefore, reference is made to surge in the numbered stations where velocities were measured, and at tide stations reference is made to seiche. For example, at Tide Station A, the notation on Figure 1 indicates that a wave or seiche with a $2\frac{1}{2}$ minute period and a maximum height of 1.4 ft. was observed and that the daily average maximum height of this wave was 0.4 ft. It also indicates that waves with periods ranging from 12 to 18 seconds were observed and that these waves had maximum heights of about 0.3 ft. A trace of waves with a period of one hour was also present. The legend for Station 1 indicates that there were fluctuations in the current with periods of about one hour. The maximum strength of the hourly flood surge preceded the peak of the seiche at the inner harbor by 18 minutes and the peak of the ebb flow preceded the low point of the seiche at the inner harbor by 16 minutes. The maximum hourly surge velocity was 0.5 knots on the ebb and 0.6 knots on the flood flow. The average velocity of the surge for the hourly waves was 0.5 knots. Oscillations with periods ranging from 2 to 8 minutes were also detected.

One very interesting fact to be noted from the results shown on Figure 1 is that most of the tide stations indicate the presence of waves with periods in the range of from 2 to 5 minutes. It is also to be noted that in this period range there is a predominance of waves having periods of about 2 or 3 minutes. This would indicate that a disturbance or wave train having approximately this characteristic period is reaching the area from the seaward side. This is contrary to the opinion of some engineers, who believe that the periods in this range are present because they represent the natural period of the various basins involved. This important question could be answered if tide gage records were available for some station on the seaward side of the outer breakwater. Unfortunately, such data are not available and they are very difficult to obtain.

It will be noted that the heights of the short period waves indicated in Figure 1 are small. The writers of the United States Coast and Geodetic Survey (USC&GS) report state that the recorded heights of these waves were undoubtedly less than the actual heights because of restricted openings in the float wells.

The report of the tide and current survey (1) contains observations of the motion of two ships berthed at Long Beach Pier A. The period of rise and fall of both ships was 14 seconds. This corresponded closely with the period of waves observed at tide stations. The report also states that during the time of the survey no damaging surge occurred in the harbor. Therefore, the much needed correlation between ship and water motion during periods of active ship motion cannot be obtained from this report. In order to delineate the problem of the model study, information of this kind is very necessary.

E. MEASUREMENTS MADE FOR STUDY

1. VISUAL OBSERVATIONS

In order to correlate the water motion with damaging ship motion, arrangements were made to obtain reports of any damage sustained by ships and drydocks because of surge action. During surge action, observations were also made by the Navy of the wave height, period and direction of motion of the waves at a number of fixed observation stations. Several of these were located at the outer breakwater where observations were made on the waves outside and inside the breakwater. Staff gages mounted on dolphins outside the breakwater and on individual piles inside the

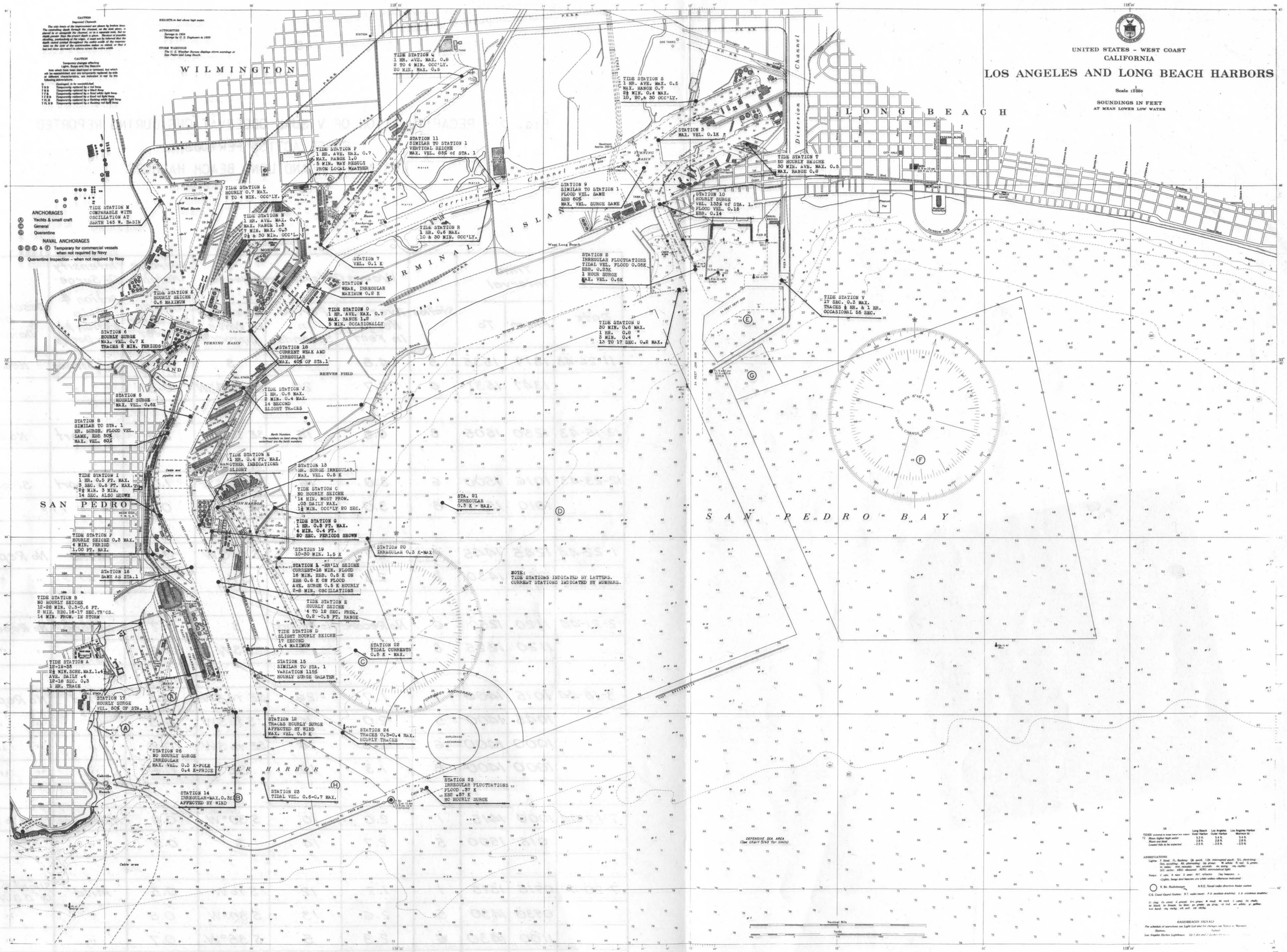


Fig. 1 LOS ANGELES AND LONG BEACH HARBORS SHOWING RESULTS OF TIDE AND CURRENT SURVEY BY US COAST AND GEODETIC SURVEY 1935-36

FIG. 2 RECAPITULATION OF VISUAL OBSERVATIONS DURING REPORTED SURGE ACTION PERIODS
LOS ANGELES AND LONG BEACH HARBORS

Date	Time Interval		Station	Waves Outside Breakwater			Average Height Of Waves Inside Brk'water In Feet	Intensity Of Surge Action & Ship Motion At Dry Docks	Description Of Damage
	From	To		Average Height In Feet	Period In Seconds	Direction Of Wave Approach			
11-26-43	1319	1535	5	4.8	21	S	0.7	Ext. Strong	Heavy
	1347	1531	6	2.7	20	SW To SSW	0.5	" "	"
12-15-43	1250	1505	8	0.8	15	SE To SW	0.9	No Report	None
12-23-43	1016	1500	6	3.0	16	SSW To SSE	1.0	No Report	Slight
	1010	1510	8	3.0	16	NE To SE	0.6	" "	"
1-22-44	1245	1445	6	2.1	24	S To SW	0.6	Mild	No Report
	1255	1430	8	2.3	15	SW	0.9	"	" "
2-21-44	1120	1250	6	No Record	No Record	S	0.7	Strong	No Report
	1120	1225	8	"	"	SSW	0.6	"	" "
3-18-44	1015	1400	1	1.7	26	SE To SW	0.4	Mild	No Report
	1045	1400	5	2.0	10	S To SW	0.5	"	" "
	1000	1400	6	2.5	14	SE To SW	0.4	"	" "
	1000	1400	8	1.3	12	SE	0.6	"	" "
7-17-44	1000	1415	1	2.0	10	S 30° W	0.6	Moderate	None To D D
	1015	1500	5	2.7	14	S	0.5	"	" " " "
7-20-44	0915	1515	1	1.7	12	S To W	0.5	Mild	Slight To Seat Of D D No. 3
	0930	1430	5	2.6	13	S 30° W	0.8	"	"
	1100	1430	6	2.3	5	S 35° W	0.7	"	"

breakwater were used in these observations. Readings were taken every fifteen minutes and recorded along with notes of other observations. These observations are summarized in Figure 2, along with the reports of intensity of surge and damage at the drydocks. The location of the observation stations is shown in Figure 3. Surges were reported on eight days during the period of the study. The surge which occurred on November 26, 1943, was the most intense and caused the heaviest damage. The other surges were considerably less serious. It will be noted that the waves outside the breakwater on November 26 were higher than during any other surge condition. The average height was 4.8 ft. However, the narrative report indicates that waves as high as 8.5 ft. were observed. Figure 4 is a photograph of the fender pile system showing damage which occurred during the surge of November 26. The battered condition of the piles and timbers indicates that the ships have been striking the pier with considerable force. Other photographs and reports show that quite a number of fender piles were broken and that many lines were parted, due to the excessive motion of the ships. Another point to be noticed in Figure 2 is the appreciable reduction in wave height inside the breakwater. Reductions of from three to six-fold occurred in every case except on December 15, 1943, where the waves inside at Station 8, which is east of the east navigation entrance, were slightly higher than outside. This peculiar situation was reported in several narrative reports and was substantiated by the staff gage measurements.

2. TIDE GAGE MEASUREMENTS.

When the model study was started in December, 1943 the Navy was operating two float type mechanical gages at the end of Pier 1. At this time one of these gages, known as the portable gage, had a time scale of 0.4 inches to the hour. The other one, known as the standard gage, was operated with a time scale of 4 inches to the hour and a vertical scale of 1" to the foot. In February, 1944 the standard gage was revised to give a time scale of 30 inches to the hour and was operated continuously at this rate. The portable gage also operated continuously but its time scale was not changed. Subsequently, float type gages were installed at several other stations. The location, type, and period of operation of these gages are shown in Figure 5. The location of all of the stations is also shown on the map of Figure 3.

In setting up these tide gage stations, it was recognized that the data of most importance to the study would be collected during times of damaging action in the drydock area. In order to make sure that these vital data were collected, it was considered advisable to operate tide stations continuously, even though in this manner considerable data not absolutely necessary for the study would be collected.

Figure 6 shows sections of marigrams at Pier 1 and in the outer Long Beach harbor area. Curves a, b, c, e, and f are for the periods during which surge was reported. Curve g is the marigram at Station 9 during February 19, 1944, which is considered

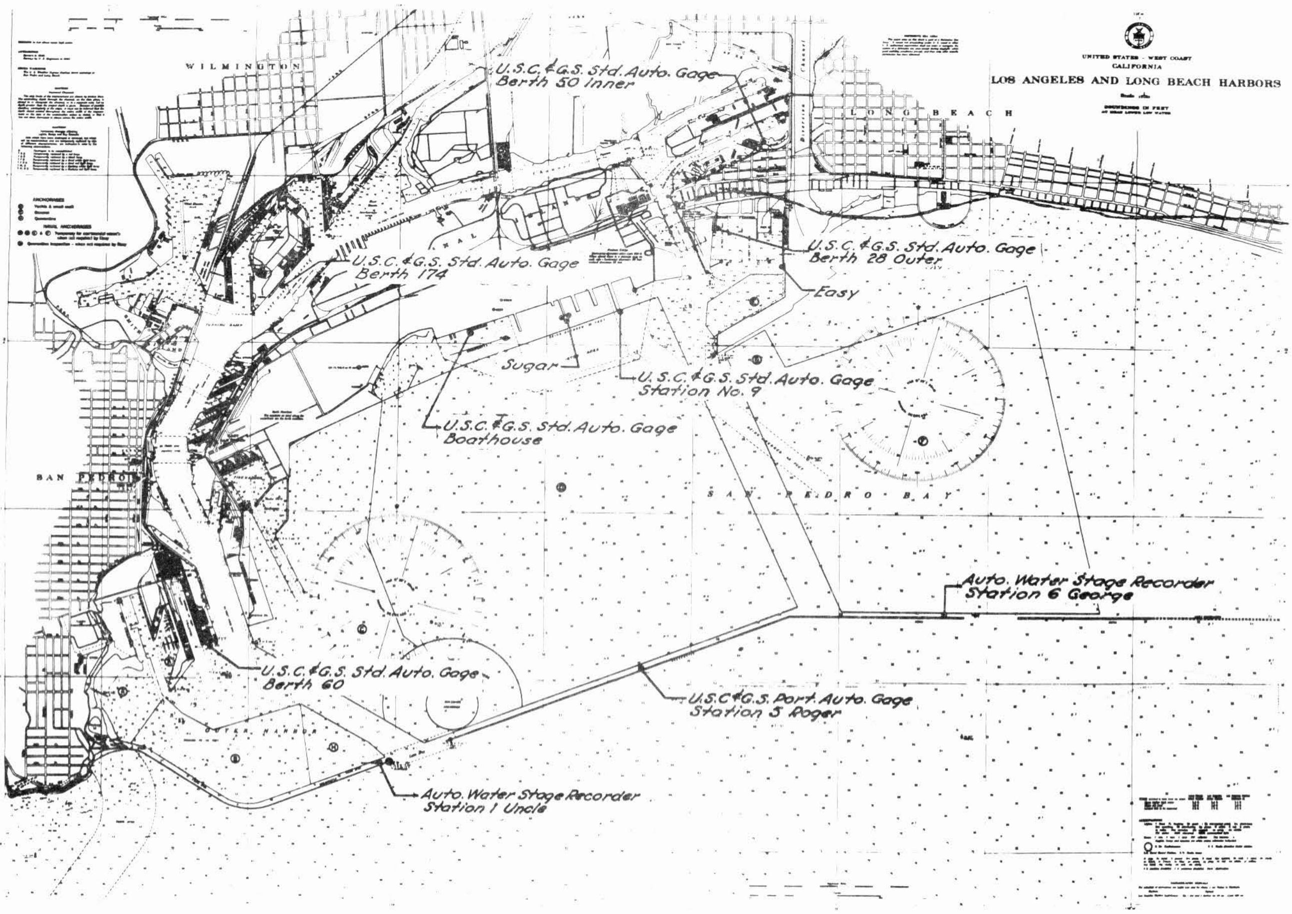


FIG. 3 CHART OF LOS ANGELES AND LONG BEACH HARBORS SHOWING LOCATION OF TIDE GAGE AND OBSERVATION STATIONS IN HARBOR AREA

FIG. 4 FENDER PILE SYSTEM DAMAGED DURING SURGE OF NOVEMBER 26, 1943

FIG. 5 DESCRIPTION, LOCATION, AND PERIOD OF OPERATION OF TIDE GAGE STATIONS

Station No. or Name	Location	Type of Recording Instrument	Well Diam	Well Opening	Elev. of Opening	Marigram Scale: Time Inches per Hr	Marigram Scale: Height Inches per Ft	Period of Operation: From	Period of Operation: To
Naval Operating Base Boathouse	S W Corner Boathouse	U.S.C.&G.S. Automatic Tide Gage (Std)	12"	3/4" Diam	-7.5'	1"	1"	9-21-42	Date
Hut No 1 (Sta. 1)	West of West Gate	Automatic Water Stage Recorder	10"	Originally 10" Diam Now 8x12" Var Port Set 1"x8"	-20.0'	1.6"	1"	2-12-44 2-23-44	2-23-44 Date
Hut No 5 (Sta 5)	East of West Gate	U.S.C.&G.S. Portable Automatic Tide Gage	4"	Originally 4" Diam Now 1¼" Diam.	-20.0'	4"	.7"	2-18-44	Date
Hut No 6 (Sta 6)	West of East Gate	Automatic Water Stage Recorder	10"	Originally 10" Diam. Now 8x12 Var. Port Set 1"x8"	-20.0'	1.6"	1"	2-12-44 2-24-44	2-24-44 Date
L.A. Harbor Primary Station	Main Channel Berth 60	U.S.C.&G.S. Automatic Tide Gage (Std)	12"x12"	1½" Diam.	-6.0'	1"	1"	Prior to 1924	Date
L.A. Harbor Mormon Island	West of East Basin Berth 174	U.S.C.&G.S. Automatic Tide Gage (Std.)	12"x12"	1½" Diam.	-6.0'	1"	1"	Prior to 3-11-31	Date
Long Beach Inner Harbor	Long Beach Inner Harbor Pier 1 Channel 3	U.S.C.&G.S. Automatic Tide Gage (Std.)	10"	1" Diam.	-9.0'	1"	1"	About 1923	Removed May 1944
Long Beach Outer	Pier A (Twice Moved)	U.S.C.&G.S. Automatic Tide Gage (Std.)	10"x10"	1" Square	-9.0'	1"	1"	Installed 1924	6-9-39
Long Beach Outer	Pier D-Berth 28	U.S.C.&G.S. Automatic Tide Gage (Std.)	10"x10"	1" Square	-9.0'	1"	1"	6-9-39	Date
Station #9 Portable	Southend Drydocks Pier 1	U.S.C.&G.S. Portable Automatic Tide Gage	4"	1" Diam.	-12.57'	.4"	.4"	9-1-43	Date
Station #9 Standard	Southend Drydocks Pier 1 in Building	U.S.C.&G.S. Automatic Tide Gage (Std) (Clock Driven)	12"	1½" Diam.	-11.95'	4"	1"	11-5-43	1-1-44 (Intermittent)
Station #9 Standard	Southend Drydocks Pier 1 in Building	U.S.C.&G.S. Automatic Tide Gage (Std) (Clock Driven)	12"	8"x12" Variable Port Wide Open	-10.45'	4"	1"	1-1-44	1-9-44 (Intermittent)
Station #9 Standard	Southend Drydocks Pier 1 in Building	U.S.C.&G.S. Automatic Tide Gage (Std) (Electric Driven)	12"	8"x12" Variable Port Wide Open	-10.45'	30" Approx.	1"	1-9-44	2-23-44 (Intermittent)
Station #9 Standard	Southend Drydocks Pier 1 in Building	U.S.C.&G.S. Automatic Tide Gage (Std) (Electric Driven)	12"	8"x12" Variable Port Wide Open	-10.45'	30" Approx	1"	2-23-44	Date (Continuous)

a fairly normal, quiet day. Curve d is for Station 9 during February 22, 1944, on which extremely high waves occurred, although no surge was reported on this day. It is easy to see that the waves with periods of about three minutes were unusually high on November 26. It is also plain that the wave conditions during surge action are abnormal. In each case of reported surge, the marigrams show abnormally high waves with periods of about three minutes. These waves are also present during the normal condition shown by the marigram of February 19, but they are quite small.

Figure 7 summarized the data on wave heights and periods obtained from the marigrams for periods of reported surge. It is interesting to note that on all of these days waves with periods of about three minutes were of appreciable height. It is particularly interesting to note that on November 26, 1943, at Station 9 and in the outer Long Beach Harbor the three minute waves were high even compared to the shorter period waves. It will be remembered that on this day the most intense surge condition occurred in the drydock area, indicating a definite correlation between height of three minute waves and the intensity of action and damage.

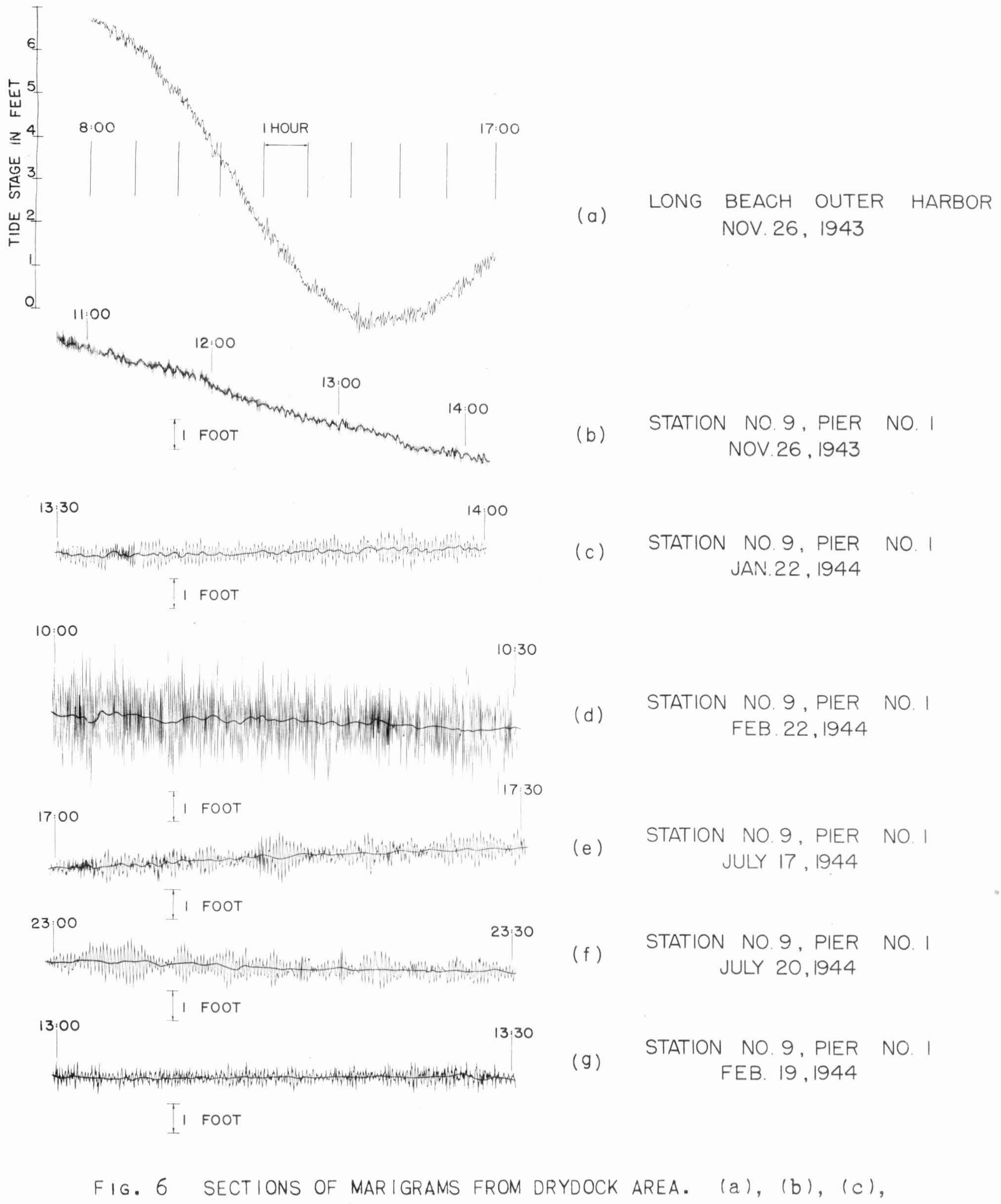


FIG. 6 SECTIONS OF MARIGRAMS FROM DRYDOCK AREA. (a), (b), (c), (e), and (f) during periods of reported surge; (d) during period of high waves with no surge reported; (g) for normal conditions.

FIG. 7 SUMMARY OF WAVE DATA FROM MARIGRAMS DURING PERIODS OF REPORTED SURGE ACTION

Station	Date	10-20 Second Period: Period Seconds	Wave Height Ft. Max.	Wave Height Ft. Ave.	2-20 Minute Period: Period Minutes	Wave Height Ft. Max.	Wave Height Ft. Ave.	30-80 Minute Period: Period Minutes	Wave Height Ft. Max.	Wave Height Ft. Ave.
Station #9	11-26-43	16to24	.58	.27	2½	.28	.13	30	.14	
End of					3	.32	.18	37	.23	
Pier 1.					4	.30	.20	48	.22	
					4½	.35	.23	53	.25	
	12-15-43							30	.10	
	12-23-43	16	.54	.30	2	.12	.10	24	.18	
					2½	.09	.09	25	.13	
					3	.19	.12	26	.12	
					3½	.12	.12	30	.22	
					4	.12	.10	33	.19	
					4½	.11	.10			
					5	.11	.10			
	1-22-44	16	2.50	.72	2	.23	.18	30	.22	
					2½	.21	.20			
					3	.24	.20			
					3½	.30				
					4	.29				
	3-18-44	7	1.71	.30	2	.16	.09	30	.11	
		12	1.32	.38	2½	.12	.08	36	.14	
		14	.96	.58	3	.13	.10	46	.24	
	7-17-44	14	1.83	.61	2	.17	.09	60	.25	
					3	.14	.10			
					4	.20				
	7-20-44	15	1.80	.80	2	.12	.09	38	.08	
		5	1.80	.80	3	.18	12	40	.25	
					3½	.20	.10	45	.30	
					4	.13				
					5	.12				
Station #1	7-17-44				2	.29	.08	30	.08	
West Gate					3	.26	.11	50	.18	
					4	.25	12	55	.10	
					5	.28	.12	60	.15	
								75	.18	
	7-20-44				2	.12	.11	30	.10	
					3	.14	.10	36	.13	
					4	.16	.11	38	.10	
					5	.16	.12			
Station #5	7-17-44				3	.20	.10	60	.35	28
Center Outer	7-20-44				3	.40	.16	30	.08	
Breakwater								36	06	
Station #6	7-17-44				2	.07	.06	40	.15	
East Gate					3	.08	.07	45	.15	.12
					4	.12	.09	53	.18	
								60	18	
								68	.15	
	7-20-44				3	.11	.08	23	.10	
					2	.10	.09	33	.10	
					5	.11	.10	55	.12	
Outer Long	11-26-43				3	.72	.28	30	.12	.11
Beach Harbor								40	.10	.10
								55	.18	.18
	12-15-43				3	.05	.03	26	.15	.07
								30	.18	.09
								38	.15	.07
								39	.17	.08
								48	.12	.08
	12-23-43				3½	.07	.05	30	.23	.10
								32	.16	.11
								36	.24	.13
Inner Long	11-26-43				3	.16	.10	26	.20	.20
Beach Harbor					4	.36	.15	30	.20	.16
					6	.09	.08	37	.10	
					8	.29	.20	47	.30	
					9	.30		53	.19	
					10	.47	.30	60	.10	
					11	.22	.20	65	.12	
					12	.12	.12	80	.35	
Inner Long	12-15-43				3	.02	.02	45	.06	
Beach Harbor					8	.10	.08	50	.23	.16
					9	.06		52	.17	
					10	.16	.15	55	.22	
					12	.08		60	.23	.15
					15	.08		65	.38	
					20	.22		70	.18	
								73	.22	
								75	.32	
								85	.16	
								90	.15	
	12-23-43				3	.02	.02	25	.30	
					9	.06		33	.27	
					10	.22	.15	37	.09	
					11	.12		40	.27	
								58	.30	
								59	.46	
								60	.48	.42
								62	.20	
								65	.38	
								70	.34	
								86	.44	
Berth 174	12-15-43				4	.08	.04	31	.07	
Los Angeles					4½	.12	.04	33	.17	
Harbor					5	.09	.04	41½	.16	
								44	.23	
								48½	.51	.45
								50½	.43	
								51½	.30	
								53	.29	
								55	.35	
								57	.69	
								58½	.23	
								59	.50	
								59½	.39	
								63½	.61	
								64	.25	
								66	.43	.24
								73	.22	
								73½	.26	
								74	.35	
								78	.26	
	12-23-43				4	.09	.03	30	.14	
					4¼	.12	.05	40	.25	.19
					4½	.15	.05	45	.43	
					5	.12	.07	50	.73	.47
								55	.50	.50
								60	.73	.40
								65	.29	.28
								70	54	
								80	.34	
								85	.61	
Berth 60	12-15-43				2	.40	.31	32	.12	
Los Angeles								35	.12	
Harbor								37	.13	.11
								40	.17	
								49	.17	
	12-23-43				2	.70	.28			
								30	.05	
								33	.18	.14
								35	.12	
								39	.13	
								44	.18	
								60	.36	

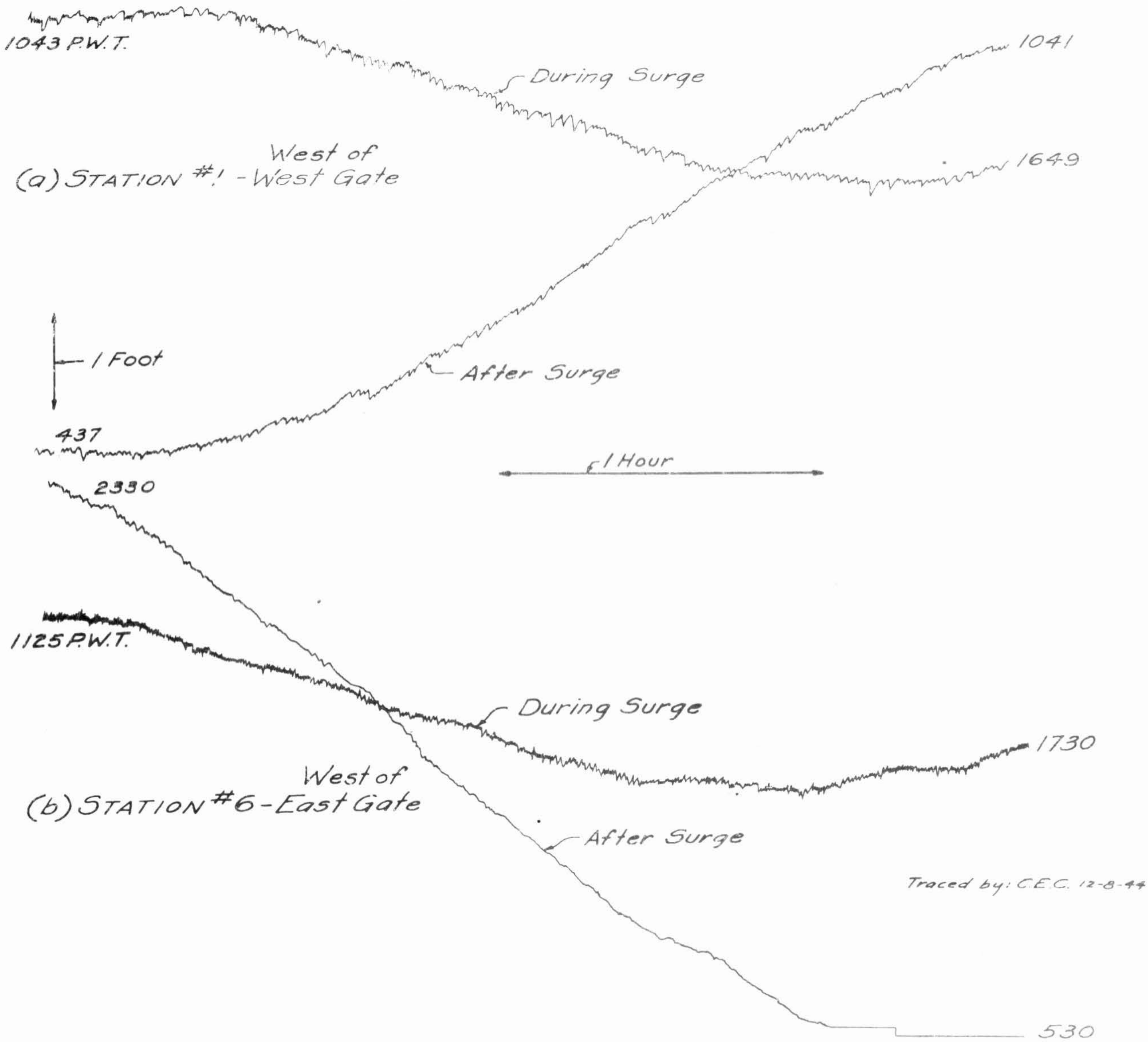


FIG. 8 MARIGRAMS FOR STATIONS 1 AND 6 DURING AND AFTER SURGE OF JULY 20, 1944

Although the action at Sta. 9, which was the station nearest the problem area, is of most interest, similar observations farther out in the harbor are also of importance. For this purpose float gage recorders were set up at stations inside the outer breakwater. Figures 8, a and b, show marigrams during and immediately following the surge of July 20, 1944, for Stations 1 and 6, respectively. In each case the marigrams after the surge followed those during the surge by from 6 to 12 hours. These marigrams indicate that the surge of this particular day was of rather short duration and subsided rapidly. By observing the marigram for the surge condition, it is possible to detect waves of appreciable height with periods of about three minutes. Waves with these periods are also present after the surge condition, but their height is very small.

In order to bring out the relationship between the waves and the ship motion and damage, Figure 9 was prepared. This chart

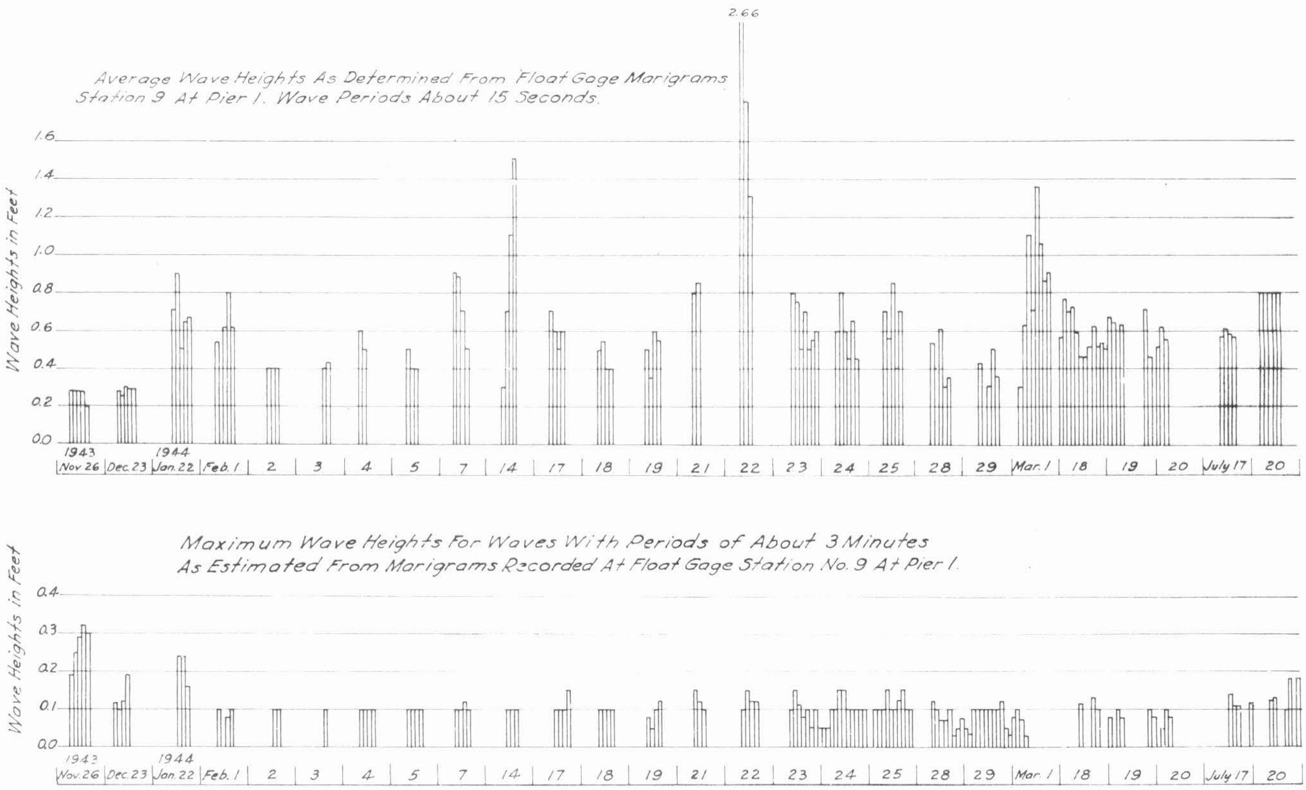


FIG. 9 HEIGHTS OF WAVES WITH THREE MINUTE AND FIFTEEN SECOND PERIODS RECORDED AT STATION 9, PIER 1

shows the heights of waves with periods of about fifteen seconds and of about three minutes during times of reported surge and for nineteen days during February and March, 1944 when no surge action was reported. Here the correlation between reported surge and maximum height of the three minute waves is very striking. It is seen that on November 26 the three minute waves were the highest of any of the surge periods. In each case where a surge was reported, the height of the three minute waves exceeded 0.1 ft. Apparently this height is in the neighborhood of the critical height beyond which damaging motion can be expected. However, the motion is probably not always severe enough to cause interruption of operations so that some of the milder surges which exceed the 0.1 ft. only slightly are not always reported. According to Figure 9, there is apparently no correlation between surge action and the height of the fifteen second waves. On February 22, 1944, extremely high waves were recorded at Pier 1 and the water was very rough in appearance. However, no surge action was reported. The ships were observed to be riding at their berths without running on the lines and work proceeded without interruption. Furthermore, on November 26, 1944, during the most severe action recorded, the 15 second waves were very low and the harbor probably appeared very calm.

3. FLOAT DRIFT MEASUREMENTS

Observations of float drift were made near the end of Pier 1 during surge conditions. These observations were made in an attempt to detect any unusual surges or currents that might explain the motion of the ships which caused damage and interruption of work. Figure 10 is a photograph of a float used in this study.

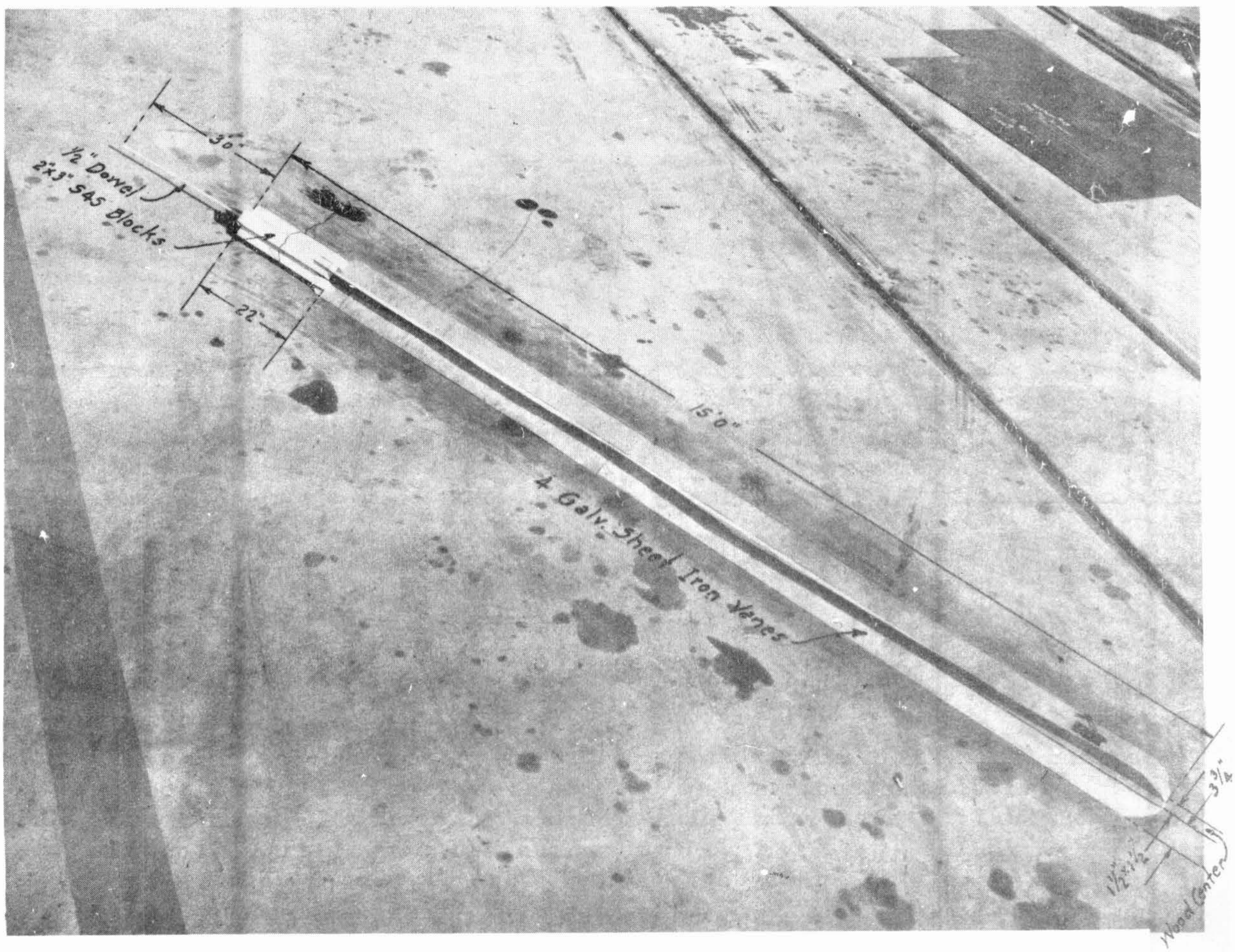
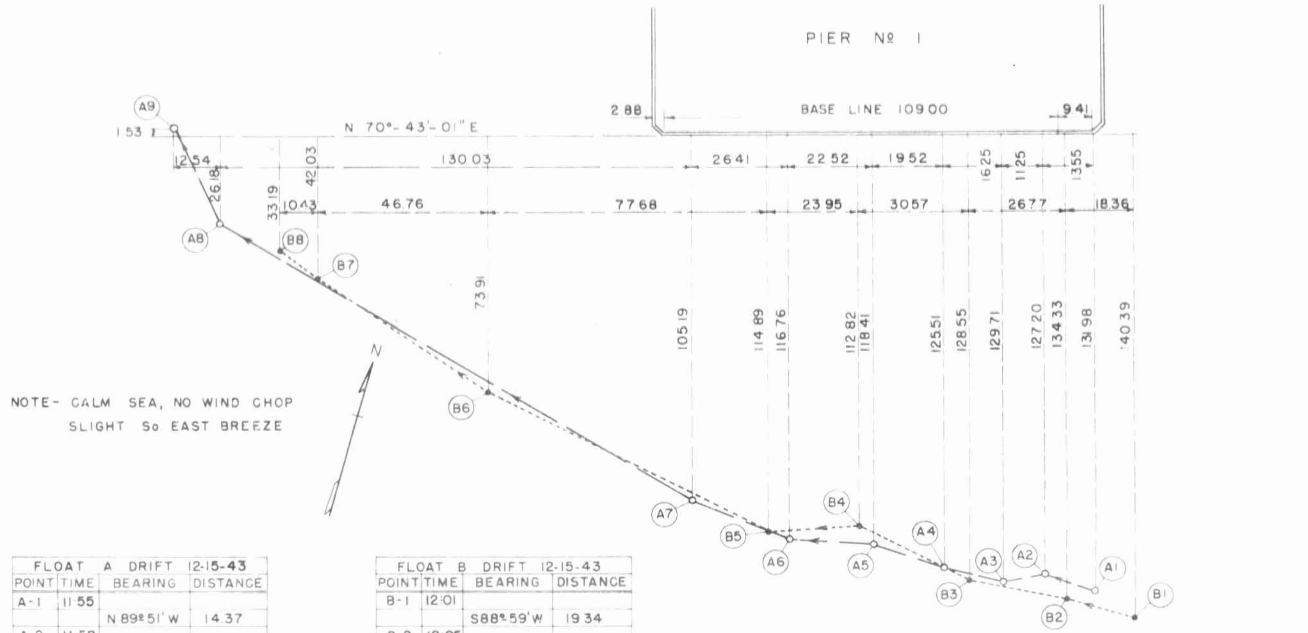


FIG. 10 FLOAT USED IN CURRENT STUDIES

It is 15 feet long and made up of four sheetmetal vanes 3-3/4 inches wide fastened together at right angles on a $1\frac{1}{2}$ x $1\frac{1}{2}$ inch wood pole. The float is weighted so that the sheetmetal portion is entirely submerged when it is in the water. A few observations of this kind were made during December, 1943 and a rather extensive series was made in January, 1944. In practically all of these observations the position of the float was determined every ten minutes. A study of the charts showing the plot of the paths does not show any systematic, periodic oscillation. Attempts to correlate the float motions and velocities with wave motions recorded by the tide gage were unsuccessful because the ten minute interval between successive observations was too long to show fluctuations with periods as small as three minutes. One set of observations made on December 15, 1943, and shown in Figure 11, was taken at from three to five minute intervals. A slight



FLOAT A DRIFT 12-15-43			
POINT	TIME	BEARING	DISTANCE
A-1	11:55		
		N 89° 51' W	14.37
A-2	11:58		
		S 58-08 W	11.53
A-3	12:01		
		S 85-12 W	16.78
A-4	12:05		
		N 89-18 W	20.77
A-5	12:10		
		S 74-54 W	22.58
A-6	12:15		
		N 85-38 W	28.83
A-7	12:20		
		N 78-00 W	152.12
A-8	12:52		
		N 43-38 W	30.41
A-9	12:57		

FLOAT B DRIFT 12-15-43			
POINT	TIME	BEARING	DISTANCE
B-1	12:01		
		S 88° 59' W	19.34
B-2	12:05		
		S 82-54 W	27.37
B-3	12:11		
		N 82-03 W	34.38
B-4	12:16		
		S 65-47 W	24.04
B-5	12:21		
		N 87-21 W	83.74
B-6	12:36		
		N 65-00 W	56.59
B-7	12:50		
		N 59-00 W	13.67
B-8	12:55		

FLOAT DRIFT
DEC. 15, 1943
11:55 TO 12:57
TRACED FROM USNDD DWG Nº 463
BY A.C.S. 12-11-44, CHECKED BY Evans

FIG. 11 PATHS OF FLOATS SOUTH OF PIER 1 DURING SURGE OF DECEMBER 15, 1943

periodic oscillation with periods of about six to ten minutes is observable in the float orbit. Oscillations corresponding with the periods of the wave trains are to be expected and their presence has been established by measurements by the United States Coast and Geodetic Survey (1). In order to measure them, the observations must be taken several times per period. Thus, to measure fluctuations with a three minute period, readings should be taken about every twenty seconds.

4. SHIP MOVEMENT OBSERVATIONS

Detailed observations were made of ship movements in an effort to get a quantitative correlation between the wave motion in the drydock area and the motion of the ships. The longitudinal, transverse, and vertical amplitudes and periods of motion of ships berthed at piers were observed every hour over extended periods, along with other pertinent information such as wind velocity and direction and damage to piers and lines. Observations of this kind were made during March, April, and May, 1944, on nine different ships berthed at Piers 1, 2, and 3. The displacement of the ships varied from 10,000 to 14,500 tons. The longest periods of observation on any one ship in one position were made on the Tanker "A" at Pier 3 and Troopship "B" at Pier 2. The results of these observations, which cover practically the entire months of March and April, 1944, are shown in Figure 12. The amplitudes of motion in this case are defined as the distance between the extreme positions rather than the distance from the mean positions to the extremes. In this chart the average amplitudes and periods for all observations in any one eight-hour period are shown and hence tend to mask the most severe conditions. However, they do give a good indication of the trends of the motion. The last two lines of this figure show, for selected intervals of time, representative heights of waves with periods of about fifteen seconds and three minutes. Figure 13 shows the data obtained from observations on Troopship "B" at Pier 2 during the period April 23 to 27. As shown by the notes at the top of the chart, on April 26 and 27 considerable difficulty was experienced with this ship and a number of lines were snapped and some damage occurred to the pier. On April 23 and 24 there was no damage and the ship motion apparently was not objectionable. It is of interest to examine the data on wave heights during the period of damage. It will be noted that there was a significant increase in the height of both the fifteen second and the three minute waves, although these waves were not abnormally large. It will be seen that the amplitudes of ship motion also increased during this period. From this it can be concluded that the increase in the ship motion was due to the increase in the wave motion.

A further examination of Figures 12 and 13 shows that the period of rise and fall of the ships is in the neighborhood of and agrees rather closely with the period of the short wave length waves and that the amplitude of this motion seems to vary approximately as the amplitude of the waves. For instance, the average of the period of rise and fall for each eight-hour interval over the two-month period varied between ten and twenty seconds and the maximum period ever recorded was twenty-three seconds, except for one seven-hour period on March 26 where the period was as high as fifty-two seconds. From this observation one is led to

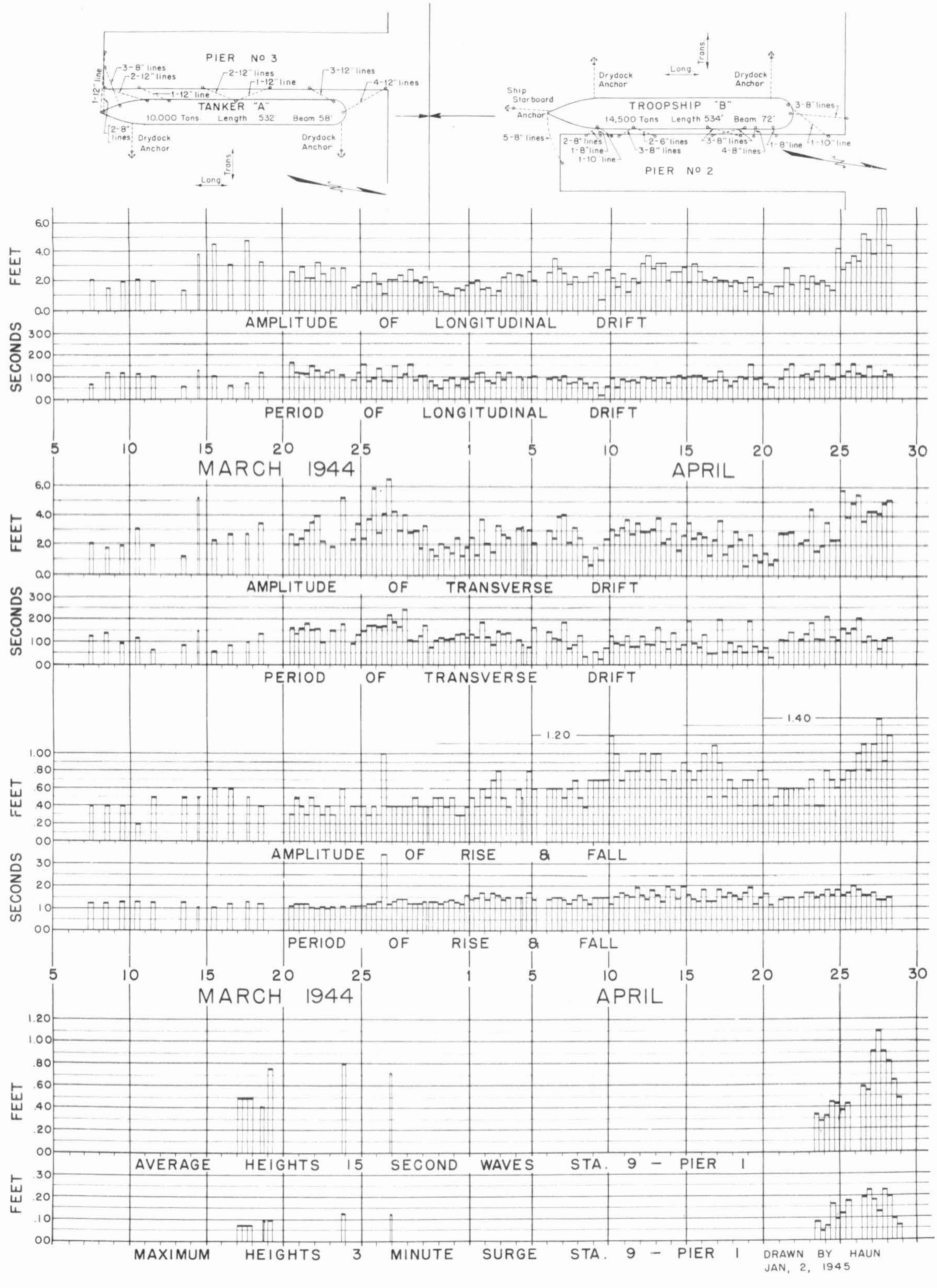


FIG. 12 OBSERVED SHIP AND WAVE MOTIONS

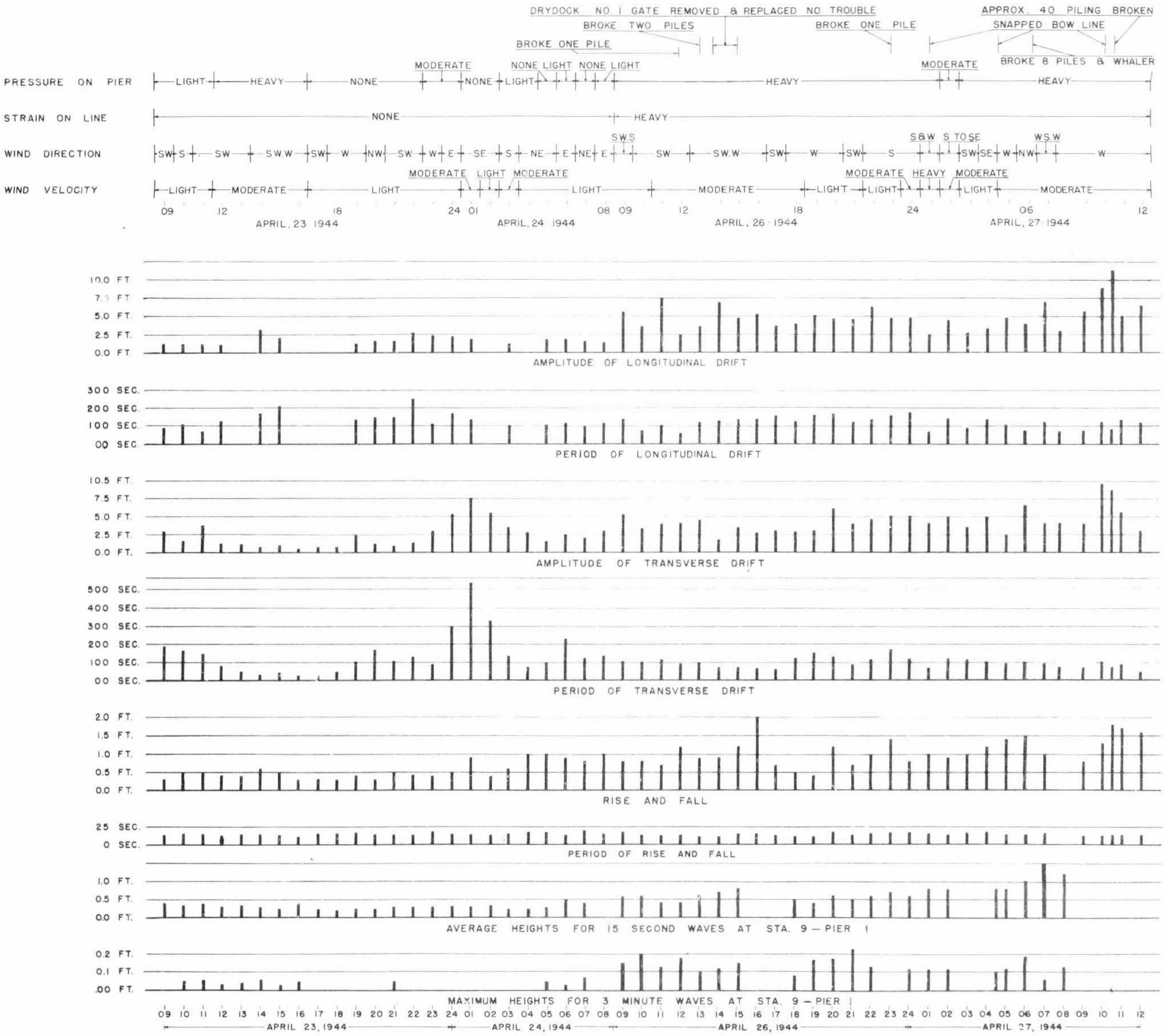


FIG. 13 OBSERVED MOTION OF WAVES AND OF TROOPSHIP "B" BEFORE AND DURING PERIOD OF DAMAGING SHIP MOTION

conclude that the rise and fall of the ship is affected predominantly by the short waves having periods of about fifteen seconds whenever they are present. This agrees with the findings of the United States Coast and Geodetic Survey (1). The fact that the rise and fall of the ship often exceeds the average height of the waves requires explanation. It is possible that the ship was pitching or rolling and that these motions contributed to the rise and fall of the point where the measurement was made.

An examination of the data on longitudinal and transverse drift shows that the periods vary approximately between one and three minutes. The amplitudes for these two motions range between one and five feet. The fact that the longitudinal and transverse drift have periods which are measured in minutes, while the rise and fall period is measured in seconds indicates that the vertical motion is excited by the short waves and that the horizontal motions, i.e., longitudinal and transverse motions, are excited by the longer waves. Since the horizontal motions are those that cause damage to piers and anchor lines, one is led to the conclusion that the waves with periods of from one to three minutes are those responsible for ship and pier damage. An examination of the wave height data on Figure 12 also shows that there is a very striking correlation between the increase in the height of waves with periods of about three minutes and ship and pier damage. These data indicate that when waves of this period reach a height of approximately 0.2 ft., damaging motion will result. This is in fair agreement with the conclusions drawn from the data presented in Figure 9 and Figure 7, which indicated that when the height of the three minute wave exceeded 0.1 ft., serious damage could be expected.

As shown in Figure 2, a mild surge was reported on March 18. An examination of the ship motion and wave data for this day shows that no damage occurred and confirms the report that this was a mild surge.

As may be seen from Figure 13, careful observations were made of wind intensity and direction concurrently with ship motion observations. A study of the wind data did not show any correlation between wind intensity and magnitude of ship motion. From this observation it is seen that the action of the water on a ship is much stronger than that of the wind and, therefore, is the controlling factor. Judging from the severity of ship motions during damaging conditions, this conclusion is reasonable.

5. AEROPLANE PICTURES

Some extremely valuable data were obtained from photographic and visual observations made from the air. Information of particular value was obtained on:

(a) Wave patterns and directions seaward of the outer breakwater
(b) Passage of waves through the navigation entrances.
(c) Configuration of waves inside the outer breakwater
(d) Passage of wave and surge disturbances through the outer breakwater.

(a) Wave patterns. Figure 14 is an air photograph looking southwest showing the wave pattern on the seaward side and waves coming through the west gate at the outer breakwater on January 22, 1944, at 0230. The wave fronts on the seaward side are about normal to the breakwater and are travelling about east northeast. The waves coming through the gate can be seen easily. They appear to have a length of about 1/4 of the gate opening, or approximately

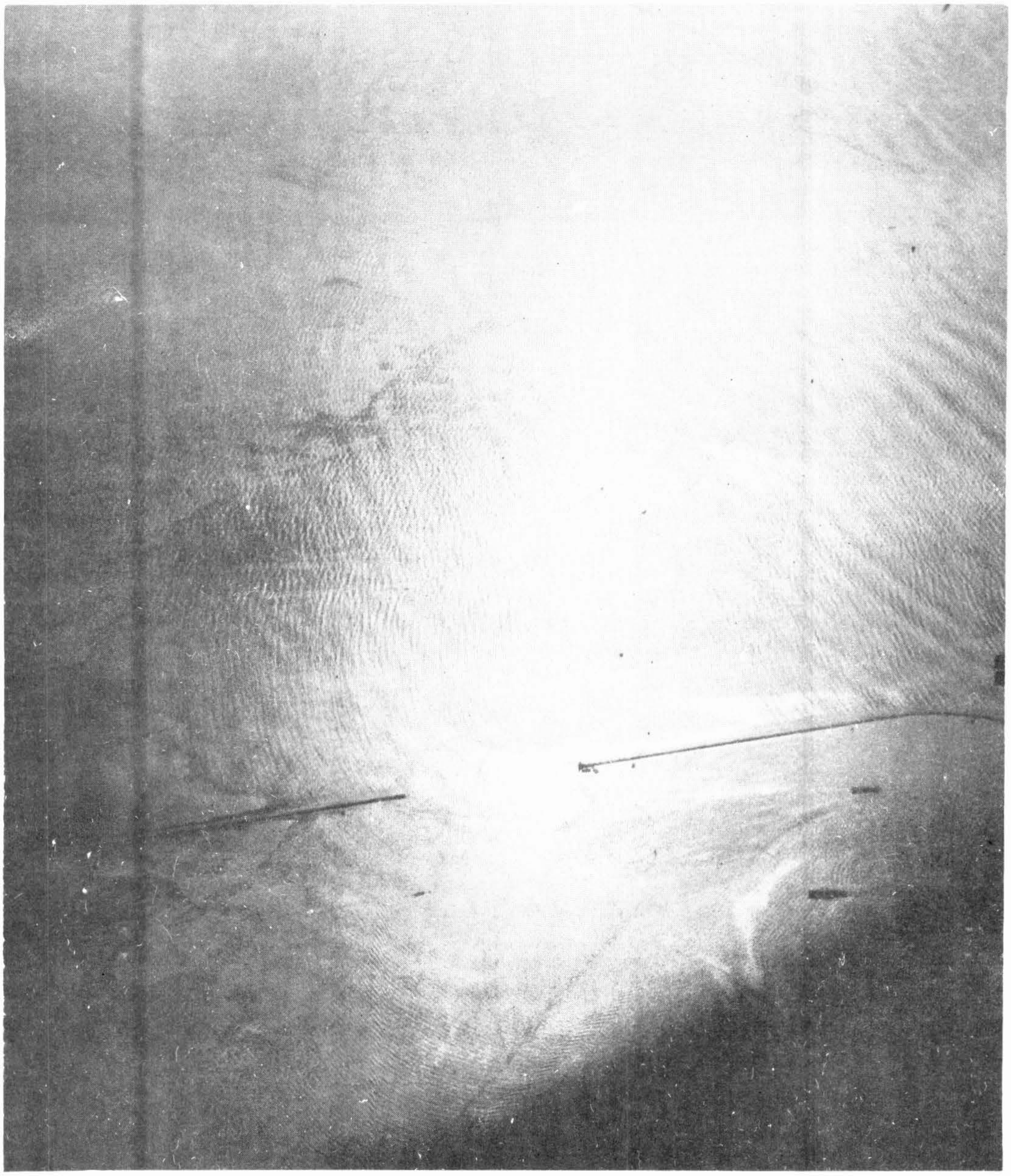


FIG. 14 AIR VIEW TOWARD SOUTHWEST FROM 12,000 FT. ELEVATION SHOWING WAVE PATTERN SEAWARD OF BREAKWATER AND WAVES COMING THROUGH WEST GATE - 1/22/44

500 ft. to 600 ft. These waves spread out in arcs with centers approximately at the gate and appear to be travelling toward the drydocks. Figure 15 shows this same area on August 16, 1944. Here again, waves of about 600 ft. wave length can be seen coming through the gate. However, the waves on the seaward side are approaching from approximately south southeast and the wave fronts

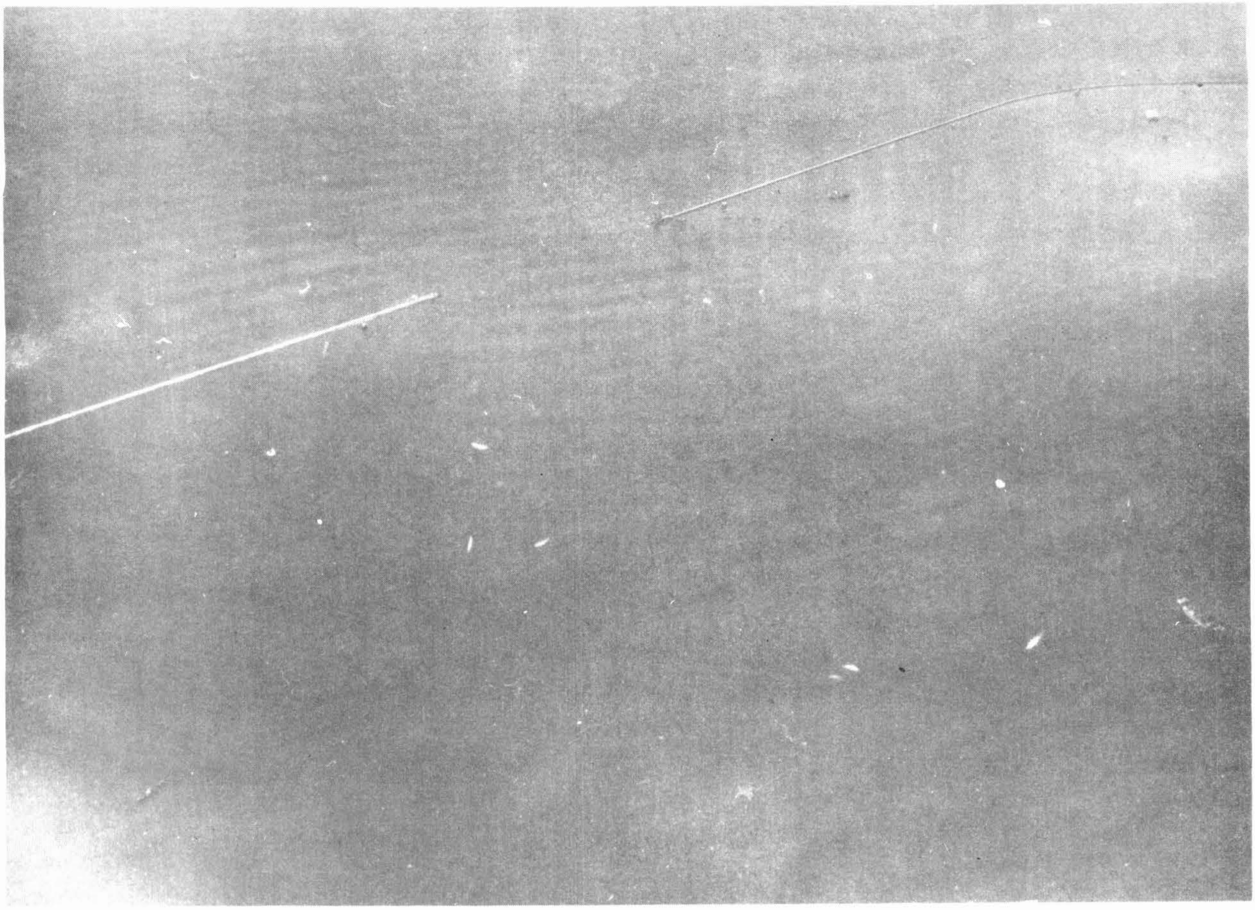


FIG. 15 AIR VIEW TOWARD SOUTHWEST FROM 5,000 FT. ELEVATION SHOWING WAVE PATTERN ENTERING WEST GATE - 8/16/44

are almost parallel to the breakwater instead of normal to it as shown in Figure 14. Figure 16 is still another photograph of the west gate taken on February 21, 1944. This photo also shows the waves coming through the gate with the waves on the seaward side coming in approximately parallel to the breakwater.

The wave pattern shown in Figure 14 occurs most frequently and is considered a normal condition. The condition shown in Figures 15 and 16 where the waves come in from the south with crests approximately parallel to the breakwater occurs less frequently and is associated with storm conditions. It will be remembered that on February 21 (see Figure 2) a surge condition was reported in the drydock area. A surge condition was also reported during January 22, 1944, although on this day the wave pattern was normal. This indicates that surge conditions can occur with either of the wave patterns.

Figure 17 is a photograph of the outer breakwater at the angle point, taken on February 21, 1944. This photograph again

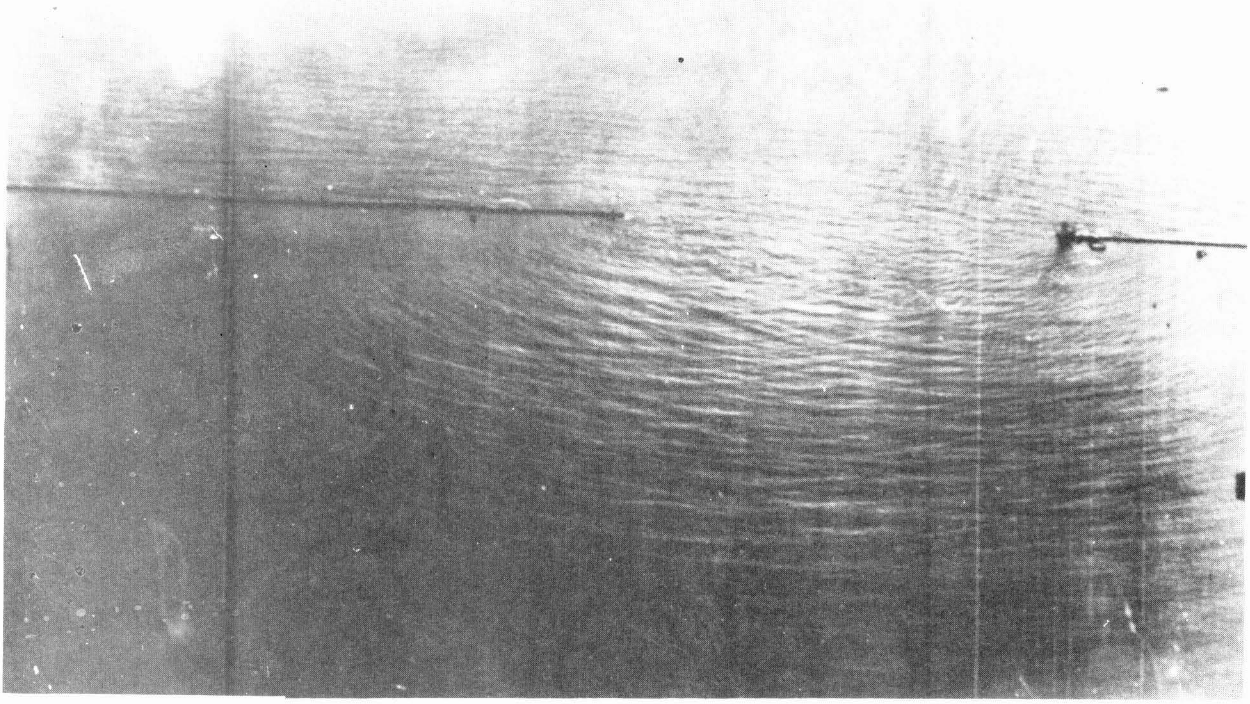


FIG. 16 AIR VIEW TOWARD SOUTH FROM 3,500 FT. ELEVATION SHOWING WAVES APPROACHING BREAKWATER FROM SOUTH-WEST AND WAVE PATTERN ENTERING WEST GATE - 2/21/44

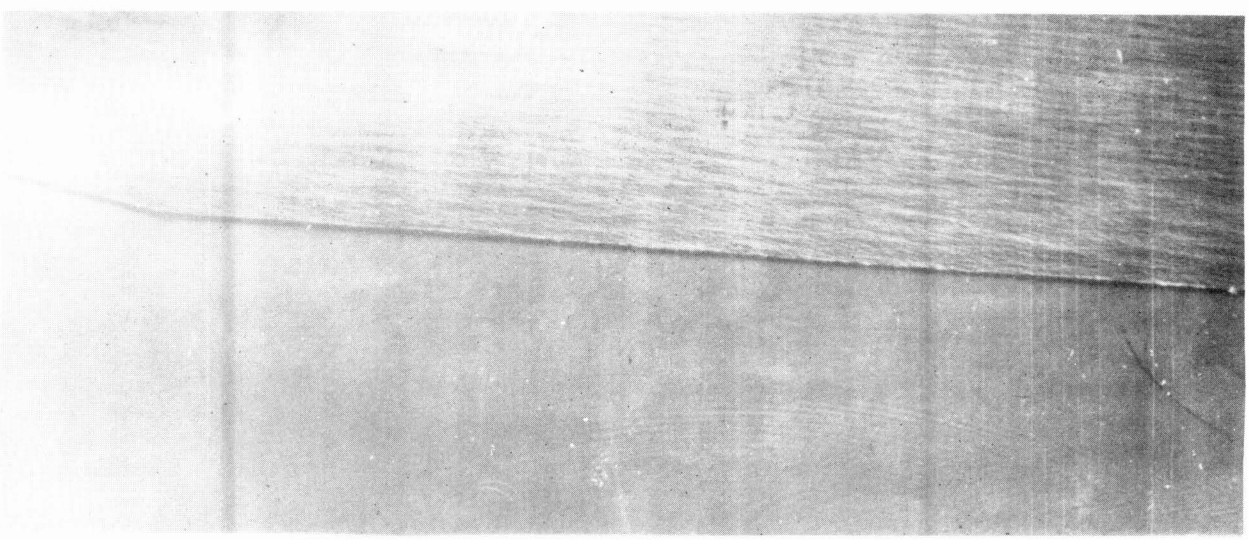


FIG. 17 AIR VIEW TOWARDS SOUTHEAST FROM 3,500 FT. ELEVATION SHOWING WAVE PATTERN AND WIND CHOP SEAWARD FROM OUTER BREAKWATER - 2/21/44

shows the waves coming in from the south. It also shows the relative calm in the harbor back of the breakwater compared with conditions on the seaward side.

Figures 18, 19 and 20 are air photographs of the east gate showing waves coming in from the south. In Figure 18 the waves can be seen distinctly coming in through the gate and in Figure 19 the waves are shown well inside the gate. In Figure 20 the waves coming into the gate are not as distinct. However, the pattern on the seaward side shows up very clearly. By comparing the wave length with the gate opening, it is possible to estimate that the wave lengths are of the order of 600 ft.

Figures 14 to 20, inclusive, show some of the characteristic wave patterns that occur in this area. Storm waves have also been observed to approach this area from the southwest. However, no photographs of this condition were available. Taken as a group, these photographs demonstrate clearly that wave energy comes into the harbor area from the seaward side.

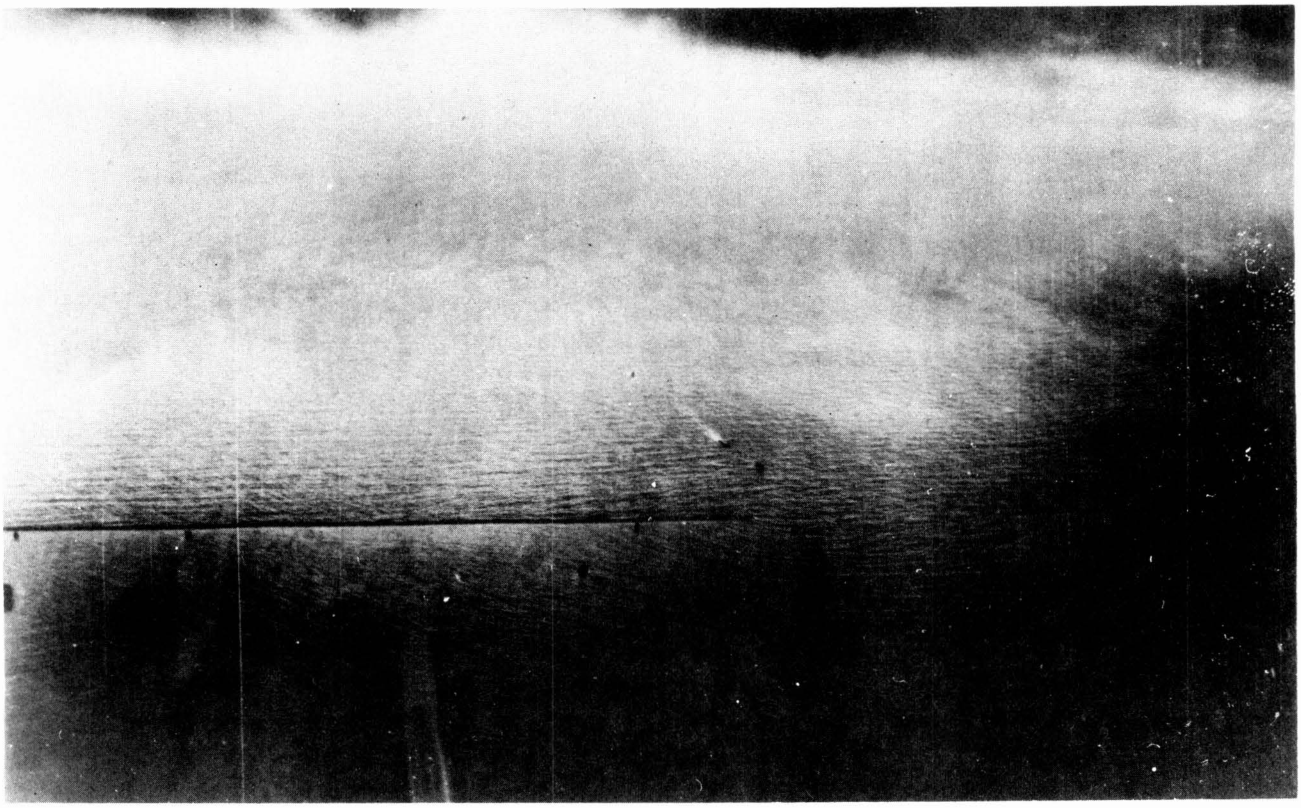


FIG. 18 AIR VIEW TOWARD SOUTH FROM 3,500 FT. ELEVATION SHOWING WAVE PATTERN ENTERING EAST GATE - 2/21/44

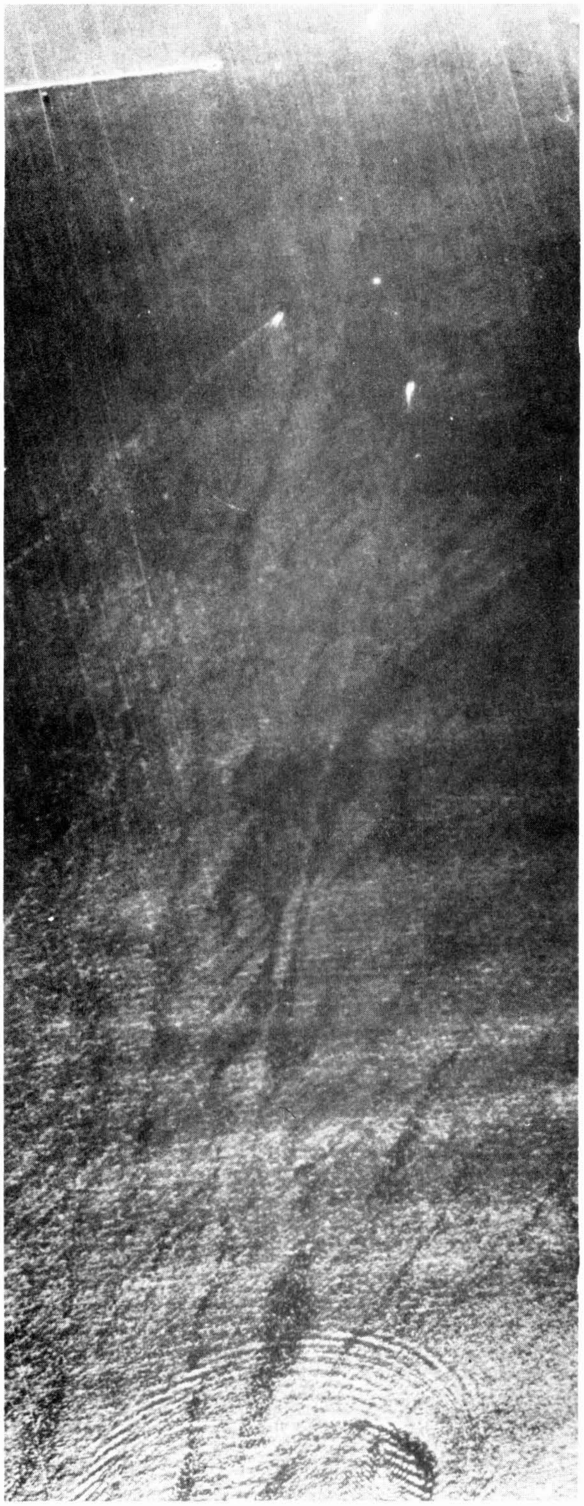


FIG. 19 AIR VIEW TOWARD SOUTH FROM 5000 FT. ELEVATION SHOWING WAVE PATTERN INSIDE EAST GATE - 8/16/44

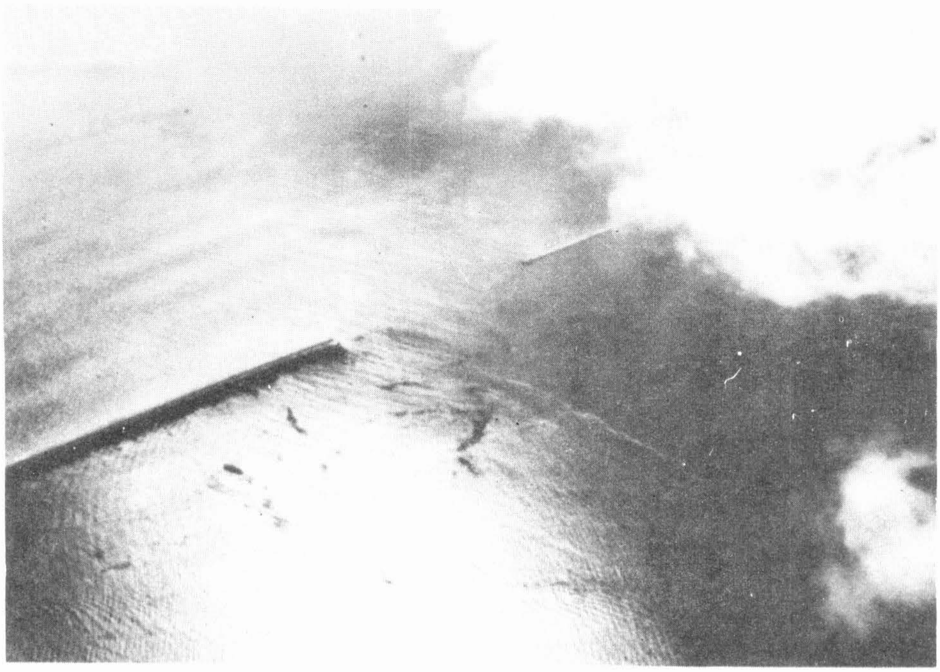


FIG. 20 AIR VIEW TOWARD SOUTHWEST FROM 5,000 FT. ELEVATION SHOWING WAVE PATTERN APPROACHING BREAKWATER FROM SOUTH AND CROSS WAVES ENTERING EAST GATE - 3/18/44

(b) Passage of waves through breakwater. Observations at the harbor from the air and from the water gave some information on the amount of wave energy that could pass through the breakwater. From the air it was possible to see white water on the shoreward side of the breakwater that corresponded to the arrival of the wave crest on the seaward side of the breakwater. Figure 21 is a view of the outer breakwater from the shoreward side showing water coming through. The "white water" on the outside is caused by the breaking wave and the flow reaching the inside causes the "white water" on the shoreward side. This demonstrates conclusively that some wave energy definitely comes through the breakwater. This was also reported in at least two official Navy reports of observations during surge conditions. Figure 22 shows typical cross sections of the outer breakwater which are at Stations 185 + 75 and 203 + 75, where the origin of stations is at the San Pedro breakwater light. It will be noted that above elevation -10 ft. the breakwater is made of large rock through which water can flow. The wave trains in Figure 21 which cause water to come through the breakwater are the short trains with wave lengths of about 600 ft. If these short period waves can cause any water to flow through, it is easy for relatively large flows to come through with waves of longer period, since the rise in elevation persists for a longer time, thus allowing more flow to occur. As an extreme example, it is obvious that there is no appreciable difference in elevation between the inner and outer side of the breakwater during the tide cycle. The tide can be

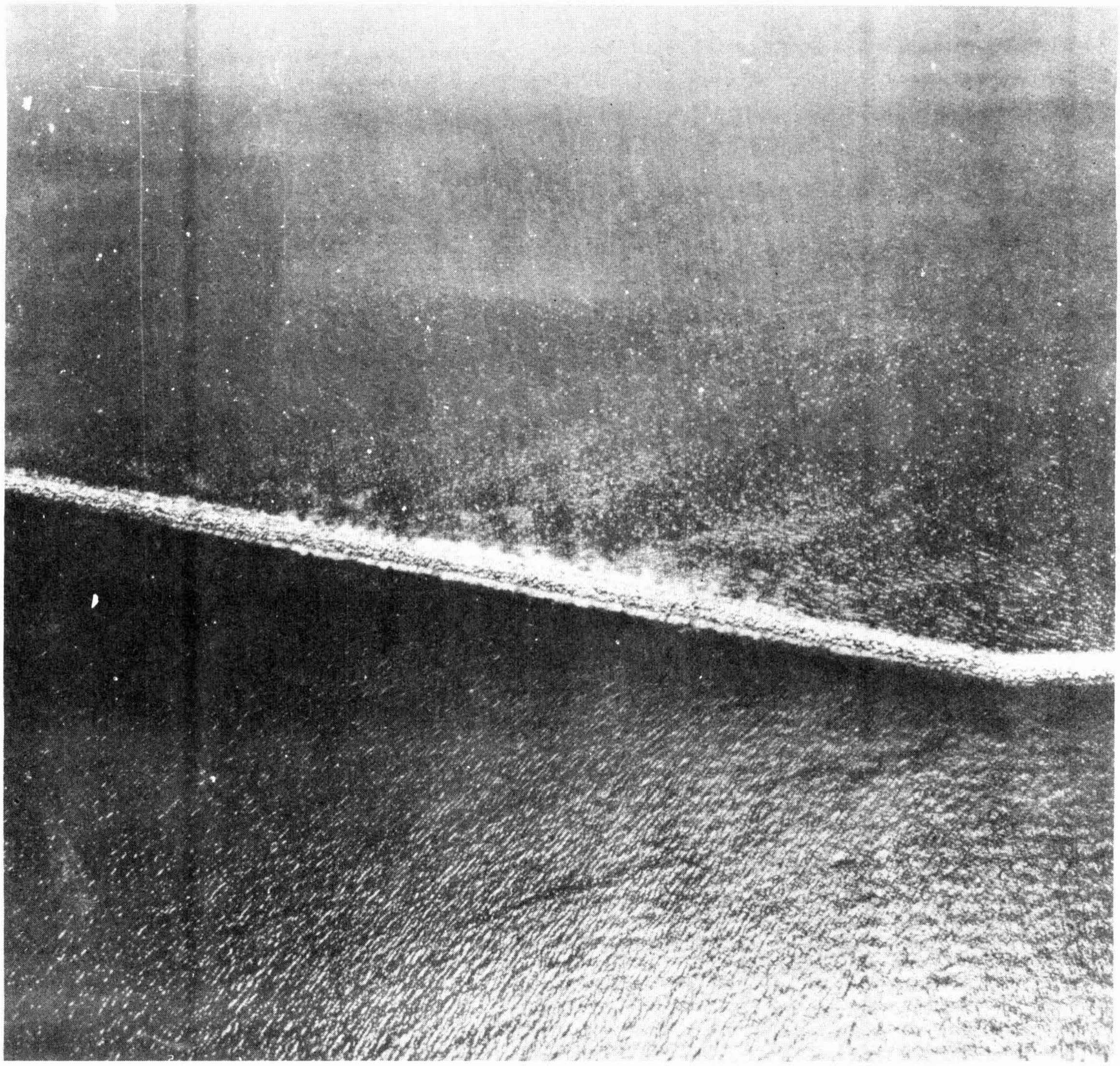


FIG. 21 AIR VIEW TOWARD SOUTH FROM ABOUT 1,000 FT. ELEVATION SHOWING WHITE WATER COMING THROUGH BREAKWATER - 8/16/44

considered as a wave with a period of about twelve hours. As the period shortens, one would expect to detect certain differences in elevation, i.e., a certain protection from the breakwater. This protection would increase as the period decreased until for the extremely short period waves, such as the wind chop, complete protection would be anticipated.

6. SPECIAL ELECTRIC GAGES

In order to get some much needed specific information regarding the waves in the harbor area, a rather extensive system of special electric and electronic gages was established in the area. These gages were procured, installed and operated by Navy

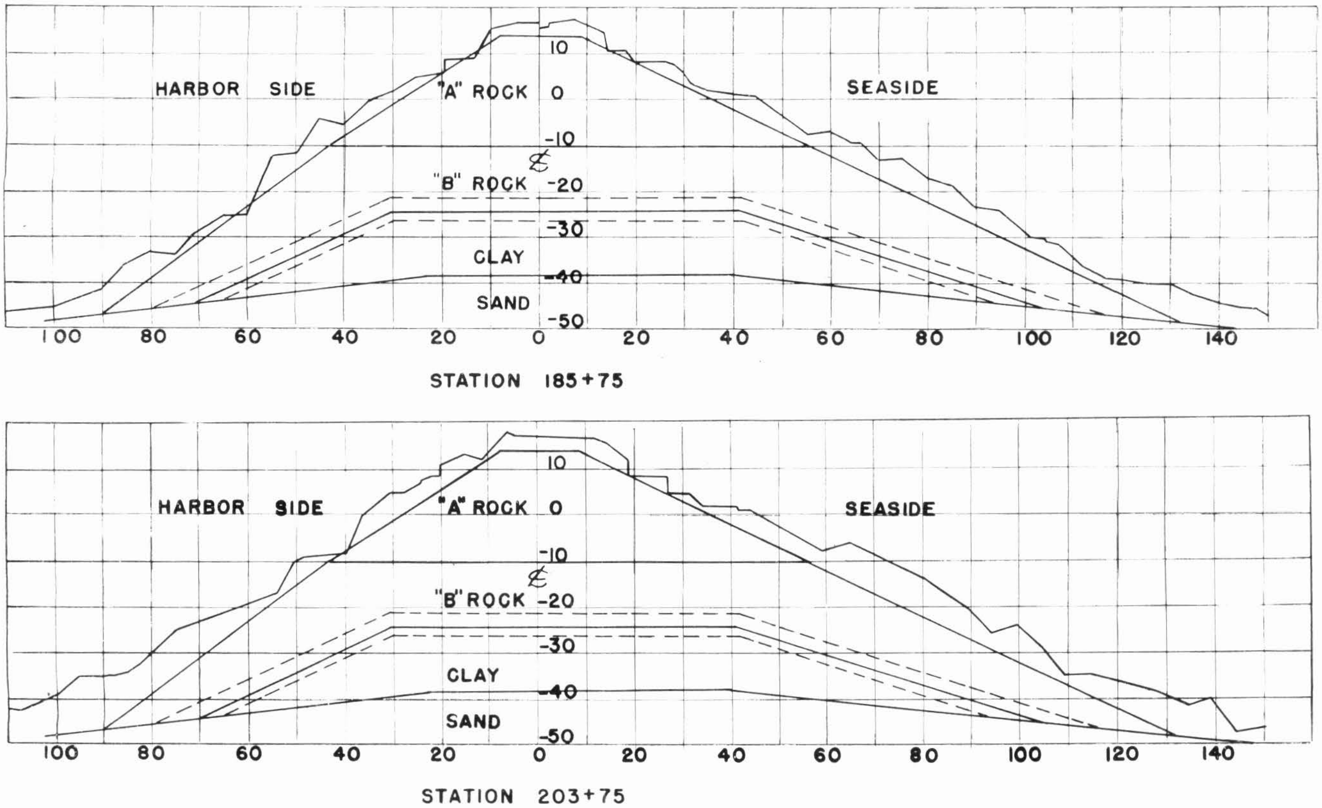


FIG. 22 TYPICAL CROSS SECTIONS OF OUTER BREAKWATER

personnel who were not under the direction or control of the Laboratory. These gages, which will be referred to as electric gages, had the advantage over the conventional float type tide gages that the units which record the rise and fall of the water surface could be installed on the bottom and the signals from these units transmitted to automatic recorders by means of electric cables. The installation points for groups of these units are indicated by the names "Sugar", "Uncle", "Roger", "George", and "Easy" shown on Figure 3. At stations "Uncle" and "Roger" groups of three units each were installed inside the breakwater at the vertices of right triangles with 300 ft. sides. At these stations units were also installed at the toe of the breakwater on the seaward side. A group of three units was also installed at station "Sugar" at the end of Pier 3. Single units were installed at stations "George" and "Easy". Because of the flexibility of these units, it was considered possible to obtain some very imlortant information from which it was hoped to attain the following objectives:

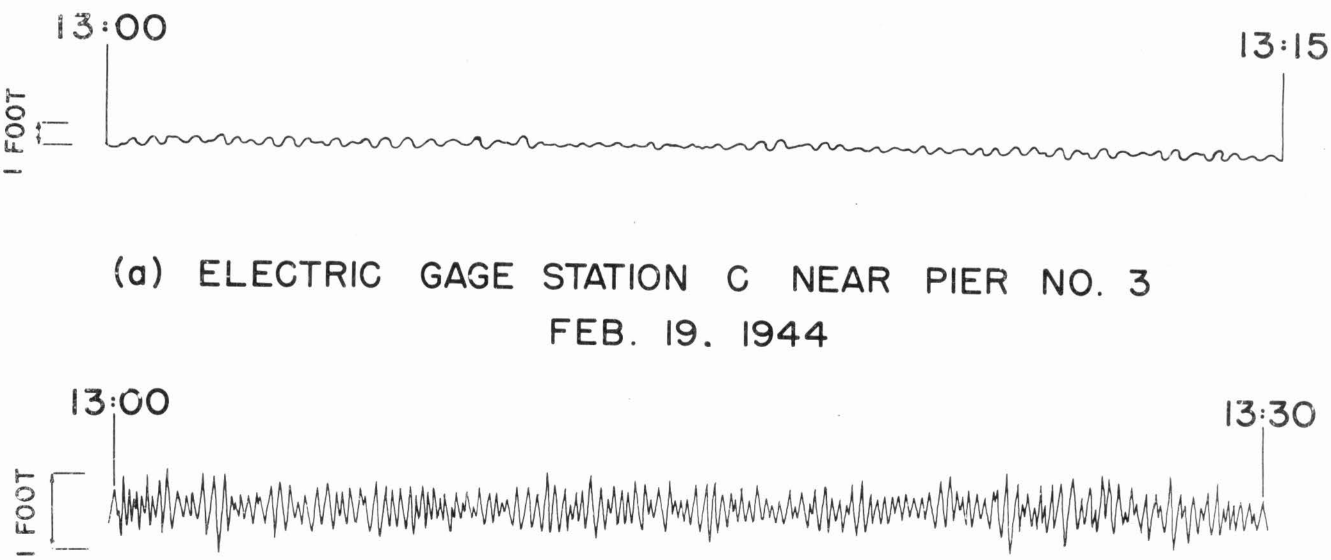
(1) Determine magnitude and pattern of waves seaward of the outer breakwater.

(2) Determine magnitude and pattern of waves coming through the navigation openings.

(3) Determine the type and amount of wave motion coming through breakwater.

(4) Determine direction of travel of wave trains at several important points in the harbor area.

Unfortunately, none of these objectives was realized. The principal reason for this was the inherent limitations of the instruments. These devices recorded only the relatively high frequency, or short period, waves. None of the records from these instruments ever showed distinct evidence of waves with three minute periods, although such waves are known to be present most of the time. Also, they did not record the rise and fall of the tide. Figure 23 is a record from the electric gage unit C at Station "Sugar" compared with a marigram from the tide gage at station 9 for the same period. It is clear from this figure that the electric



(a) ELECTRIC GAGE STATION C NEAR PIER NO. 3
FEB. 19. 1944

(b) TIDE GAGE STATION 9 PIER NO. 1
FEB. 19, 1944

DRAWN By: Evans
Dec. 12, 1944

FIG. 23 TYPICAL SIMULTANEOUS MARIGRAMS FROM ELECTRIC GAGE AND FROM STANDARD TIDE GAGE.

gage does not record all of the waves present in the harbor. An analysis of their operation showed that their damping characteristics were such that it was impossible for them to respond to wave motions having periods greater than a fraction of a minute in length. Apparently the instruments were designed for an application where the characteristics desirable for a wave gage were unnecessary or undesirable and the attempt to use them as a wave gage was a fundamental error. The heights of even the short period waves measured with these instruments did not agree with those obtained from the standard tide gages.

The arrangement of units in triangles at stations "Sugar", "Uncle" and "Roger" was intended to determine the direction of travel of waves. The direction could be determined if the differences in time of arrival of a given wave at the three units could be measured. In order to do this, it was necessary to have the time scales of any group of three units synchronized so the time intervals could be measured to a fraction of a second. Although precise synchronization was requested, it was not obtained and, therefore, such determinations of wave direction that were attempted gave very erratic results that could not be accepted.

Because of these instrumental difficulties and operating defects, it became obvious that the data from the electric gages could not be used and, therefore, the analysis of these data was dropped.

III. PLAN OF MODEL STUDY

In planning the steps required to carry out the model study it was recognized that some preliminary information would have to be obtained before the study of the basic problem could be started. Primarily, it was necessary to establish a set of standard test conditions for severe wave motion in the Operating Base area, both for the short wave length (approximately 500 ft.) trains and the long wave length (approximately 6000 ft.) "surges". To establish these conditions it was necessary to determine such factors as the direction and relative magnitude of the wave trains reaching the Operating Base area from the three breakwater openings. Similarly, the modifying effect of the outer harbor on these wave trains had to be ascertained. To answer these and similar questions two preliminary studies were started simultaneously.

A. PILOT STUDIES

In order to provide a rapid qualitative means of exploring the problem, a "ripple tank" was constructed. The ripple tank is a device in which very small scale studies of surface wave phenomena can be carried on quickly and easily. Its use in this problem was anticipated to be as a guide or pilot for the studies in the main model basin. Such pilot studies are useful in pointing out profitable lines of development and eliminating those which are clearly unpromising. They also give a preliminary picture of the over-all conditions to be encountered and some of the operating problems to be dealt with in the larger scale model.

B. PRELIMINARY STUDIES IN MODEL BASIN

The second project that was started immediately was the first model in the main basin. This was constructed on a sufficiently small scale to permit the modeling of the entire Los Angeles harbor and adjacent areas from Point Fermin to Sunset Beach. The purpose of this model was basically as outlined in the beginning of this section; i.e., to establish the conditions for the final comparative tests of the mole and the allied constructional features. To do this it was proposed to investigate the following factors:

(1) The general wave pattern in the outer harbor.

(2) The relative importance of the west gate, east gate, and open end of the breakwater as sources of disturbance in the Naval Operating Base area.

(3) The effect of the wave direction in the open ocean outside of the breakwater on the magnitude and configuration of the disturbance pattern in the outer harbor.

(4) The comparative effectiveness of different mole alignments and the relative desirability of different locations of the entrance to the enclosed area.

(5) The evaluation of the probable effect of using varying degrees of model distortion on the reliability of the study results. Note that the term "model distortion" in this connection means the increasing of the vertical scale with respect to the horizontal in order to increase the water depth in the model.

C. FINAL STUDIES IN MODEL BASIN

The final series of models contemplated for the main basin were all planned for a single scale so chosen that the area to be enclosed by the new mole would be as large as possible consistent with satisfactory boundary conditions for the study. The basic plan contemplated the installation of moles of various configurations in the model, together with selected structures, and the subjection of these combinations to a series of standardized test conditions of waves and surges. It proposed to determine the relative effectiveness of these various configurations by comparing the behaviour of the area enclosed by them with that of the same area in its unmodified condition. The following are some of the specific items it was desired to investigate:

(1) The amount of protection afforded by the mole if constructed as it was originally designed.

(2) The best performance that could be obtained by acceptable modifications of the original design.

(3) The natural frequencies of the new basin formed by the mole, with the view of comparing this frequency response with the probable frequencies of the waves and surges existing in the harbor, to evaluate the possibility of the occurrence of undesirable resonance phenomena.

(4) The location, size, and shape of the navigation gate in the mole.

(5) The ship movements at piers in relation to the vertical and horizontal water movements at the same location, together with the amount of reduction of these movements obtained by the construction of the mole.

(6) The effect on the water movement within the mole of the installation of various proposed structures in the completed basin.

IV. GENERAL PHYSICAL BACKGROUND

In order to provide a background for the discussion of the results of the various portions of this study, it seems desirable to include a brief discussion of some of the known physical factors involved in the problem at the harbor.

A. NATURE OF THE PROBLEM

All the disturbances which have been observed to cause the present difficulty in the area of the Naval Operating Base fall in one class. The disturbances enter the area either as solitary waves or as trains of waves and, therefore, the problem as a whole is one of wave mechanics. A satisfactory solution of the problem must of necessity bring about the elimination, or at least the reduction to a tolerable magnitude, of these wave trains within the working area. There appear to be only two possible approaches to this solution: (1) the provision of methods for preventing the wave trains from entering the critical area, and (2) the establishment of conditions which will ensure the damping out of such wave trains if they are permitted to enter the area. These statements and this study are not concerned with the cause and origin of the waves that produce the disturbance in the area. Aside from such obvious causes as wind and storm, it has been suggested that they may be the results of earthquakes or submarine explosions, or that sudden changes in barometric pressure may start the outer harbor basin oscillating at its fundamental period or some low harmonic of it.

B. WAVES TYPES

The surface waves that can cause disturbances of sufficient magnitude to be important in this problem are gravity waves. For the purpose of this study gravity waves can, in turn, be separated into two general classes: (1) deep water or "short" waves, (2) shallow water or "long" waves. These two types of wave trains have quite different characteristics. The principal ones that concern this problem involve the relation between the depth of water, the wave length and the velocity of the wave. In the case of deep water waves, the wave velocity is a function of the wave length and varies as the square root of this length. This velocity is independent of the depth of the water. In the case of shallow water waves the conditions are exactly the opposite. The wave velocity is independent of the wave length. However, it is a function of the depth, varying directly with its square root. In between these two distinct types of wave trains there is an intermediate, or transition, type in which the wave velocity is a function both of the wave length and the water depth. For the clarification of this classification, the fundamental equation for the velocity of a wave of very small amplitude (3) is helpful. This is:

$$C^2 = \frac{gL}{2\pi} \tanh 2\pi \frac{h}{L} \qquad (1)$$

In this equation C = Velocity of wave
L = Wave length
h = Mean water depth
g = Acceleration of gravity

It will be noted that when the depth, h, is large with respect to the wave length, L, that the hyperbolic tangent approaches unity as the limit. In this case, the expression for the wave velocity reduces to

$$C = \sqrt{\frac{gL}{2\pi}} \qquad (2)$$

This is the expression for the velocity of a deep water or "short" wave. It will be seen that such waves are called short because in general their wave length is small in comparison to the depth. Therefore, this term has nothing to do with their absolute wave length. However, it should be noted that, in water of a given depth, the deep water waves always have shorter wave lengths than do the shallow water waves.

When the depth becomes small with respect to the wave length, the value of the hyperbolic tangent approaches that of the angle and, therefore, in the limit the expression for the velocity becomes

$$C = \sqrt{gh} \qquad (3)$$

This is the well known expression for the velocity of a shallow water wave of small amplitude. Figure 24 shows a comparison of the velocities of these wave types. It will be observed that when the depth exceeds one-half the wave length the wave behaves as a true deep water wave; whereas when the depth is less than one-twentieth of the wave length, the velocity is unaffected by the length of the wave and is completely controlled by the depth. In other words, it is a true shallow water wave. This is a rather severe classification; if slightly larger deviations from the "typical" performance are tolerated, it can be stated that all wave trains whose wave lengths are ten times the depth or greater behave as shallow water waves, whereas those whose wave lengths are less than four times the depth behave as deep water waves. In this classification it should be noted that the arch-type of the shallow water wave is, of course, the solitary wave.

This classification has a direct bearing on the wave types encountered in the present study. A reference to Section II on Field Observations shows that the minimum wave length of wave trains which enter the harbor is about 600 ft. The average depth of the water at the breakwater is about 50 ft. This depth decreases as the shore is approached. It will thus be seen that the shortest wave train entering the harbor has a wave length of about twelve times the depth. This is the average wave length of

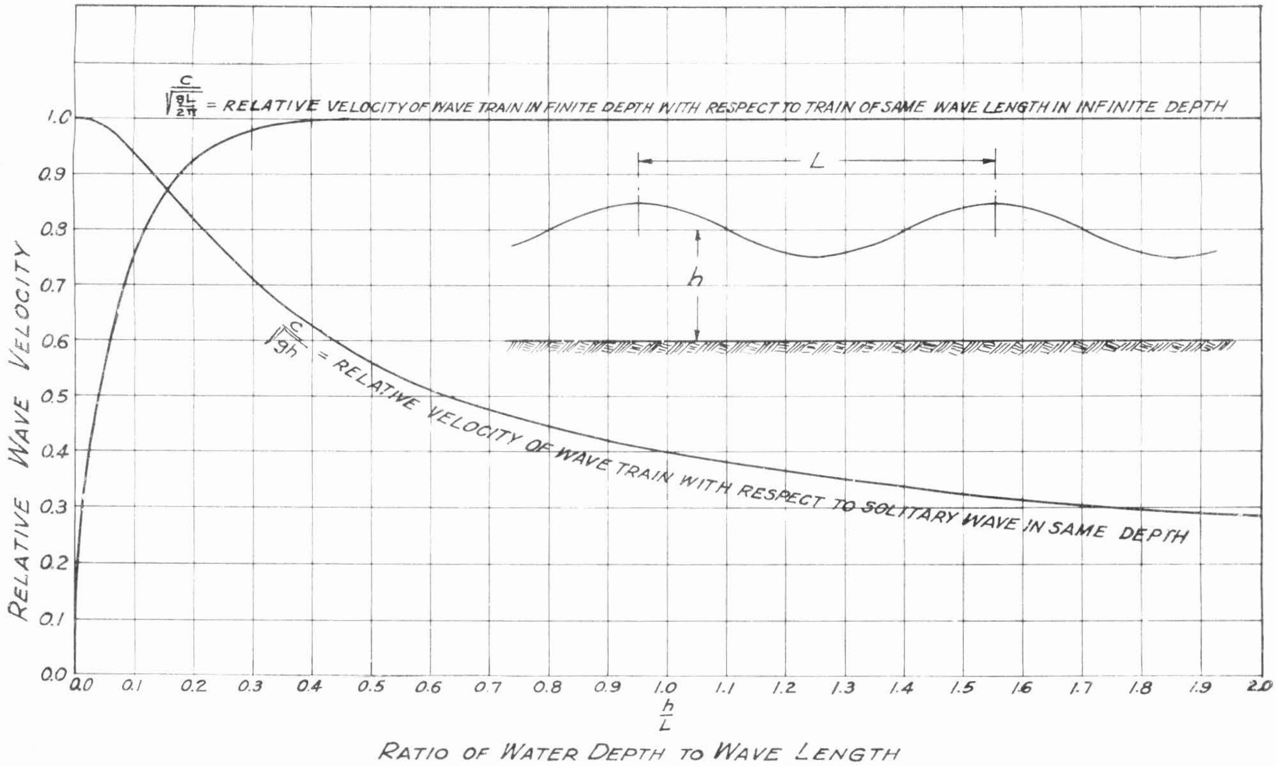


FIG. 24 RELATIVE VELOCITIES OF DEEP AND SHALLOW WATER WAVES

the normal wave train having a period of from twelve to twenty seconds. On this same basis, the wave length of a three minute surge train would be about 144 times the maximum depth. Thus, it will be seen that shallow water waves are the only type that are encountered in this problem.

C. WAVE REFLECTIONS AND EFFECT OF OBSTRUCTION

One group of wave phenomena which concerns the interaction of wave trains with various harbor construction features has a common controlling variable. This common factor is that the nature of the phenomena is greatly affected by the ratio of the length of the units of the solid boundary to the wave length. In this class will be found such phenomena as the reflection of waves from breakwaters and moles, passage of waves through gates, etc.

Consider the case of the reflection of a wave hitting a shore. This can be compared to a beam of light hitting a solid surface. If, in this latter case, the solid surface is rough, each little element of the surface will reflect the portion of the light that hits it and the angle at which the light leaves the surface will depend upon the angle of that particular surface element. The result is that the rough surface scatters the light and there is little or no reflected beam. However, if that same surface is polished until all the irregularities are so smoothed that the deviations from a true surface are less than the wave length of the light, then the surface becomes a good mirror and the light beam is reflected from it in accordance with the well-known optical principle that the angle of reflection equals the angle of incidence. The same thus holds for the reflection of an ocean wave from the shore. If the irregularities of the shoreline are small in comparison with the wave length, then the wave will be reflected in the direction accurately predicted from the direction of travel of the incoming wave and the angle of the shore. If the shoreline is a breakwater or mole constructed of loose rocks, it would appear at first glance that this would be a rough surface, but when it is remembered that in the harbor the shortest significant wave length is 600 ft., it will be seen that in comparison to this the rocks are very small and, therefore, such a shoreline would be a good reflector and little distortion should be expected in the reflected wave train.

It is possible that when a wave hits the shoreline it will be damped. That is, the amplitude of the reflected wave train will be reduced below that of the incoming train. Damping, however, is synonymous with loss of energy. The breakwater or mole, or other shoreline construction having a fairly steep slope, has very little opportunity to absorb energy from the waves. In other words, it is a good reflector and the amplitude of the reflected wave train is very little smaller than that of the initial train. The most effective means of damping waves of this order of wave length is a sloping beach which causes the wave to break and thus dissipates the kinetic energy by transferring it ultimately into heat through the medium of viscous friction.

It would be possible to build a breakwater or mole that would be a good energy damper at least for a narrow range of wave lengths. This could be done by making the mole porous enough to allow an appreciable flow to take place through it as each successive wave caused a raising and lowering of the water level on the mole. The passage of the water back and forth through the interstices would dissipate energy through fluid friction in the same manner that it does in a pipe line. However, this would be a rather delicate design problem; for if the mole were too porous, the wave would pass through it with little damping; whereas, if the mole were not porous enough, the wave would be reflected with little loss of energy.

The passage of waves through gates or around obstructions is also affected by the relative magnitude of the object and the wave length. For example, if a wave train hits a small obstruction such as a group of piles, it passes around them and a short distance behind the obstruction there is no visible effect left on the wave train. On the other hand, if the same wave train is intercepted by a good sized island, the disturbance produced on the wave train persists for a long time. This same effect is also very noticeable at gates in breakwaters and moles. If the gate opening is large in comparison with the wave length, then a wave train of that width is admitted through it without appreciable modification. On the inside of the gate the train continues essentially straight while a circular pattern develops from each corner of the gate with the corner as the center. Since the energy in the curved sections spreads rapidly, this part of the wave pattern is decreasing in intensity much faster than the straight section. This causes energy to be transferred from the straight to the curved sections. This gradually transforms the pattern into circular arcs. The wider the gate the farther away the straight part of the pattern can be distinguished. The amount of energy that comes through the gate is that brought in by the width of wave train admitted. If, however, the wave length is long in comparison to the width of the gate, conditions are quite different. In this case an appreciable difference in elevation is maintained across the gate and this difference produces a flow since, if the wave length is long, the period also must be long. This flow may be very appreciable and if the basin is relatively small, this flow may create serious currents. A typical example of this type of wave is the tide. If it were desired to construct a mole to protect a mile or two of shoreline from the movements produced by the tide, it is quite obvious that the only way of accomplishing this would be to construct an impervious mole and provide it with locks for ingress and egress. It is equally obvious that if the locks were to be removed, the impervious mole would protect the enclosed basin from the effects of, say, the 600 ft. waves, even though the amplitude of these short waves were the same as that of the tide. The reason for this is that although each length of wave may produce the same instantaneous difference in elevation across the gate, the short period waves do not maintain this difference long enough to produce an appreciable flow, whereas the tide maintains the difference for so long a time that only a complete closure can prevent the water inside the basin from following the motion of that on the outside. If this chain of reasoning is pursued, it leads to the statement that the longer the wave length of the wave train approaching a gate, the greater is the difficulty of preventing it from passing through and the larger is the energy flow through the gate into and out of the basin. This discussion implies that the wave crest is parallel to the gate opening. It is equally true if the wave crest is at an angle to the opening. In the latter case, it is easy to see that if the gate has a width of several wave lengths, there is no effective difference in level across the gate, since there will be several crests and troughs existing simultaneously in the opening.

D. STANDING WAVE PATTERNS

When a continuous wave train impinges on, and is reflected by, a shoreline such as a breakwater, mole or other good reflecting surface, the reflected wave train passes through the oncoming one. In general, the direction of the oncoming wave travel will not be normal to the reflecting surface, hence, the reflected wave train will be passing through the initial one at an angle. The result will be that an interference pattern will be set up. If two peaks coincide they will reenforce and thus increase the maximum amplitude. If a peak coincides with a trough, their motions will cancel. Since the wave lengths of the two trains are the same, the points where reenforcement or cancellation takes place will be fixed. In other words, the interference pattern will be stationary. This is normally called a "standing wave pattern". It must always exist under such circumstances. This means that there will be a standing wave pattern off shore from any land unless the land is bounded by beaches which effectively damp out the wave trains and eliminate the reflection. Such a standing wave pattern exists to the seaward of the Los Angeles breakwater. Likewise, a standing wave pattern has always existed in the area of the Naval Operating Base whenever any wave trains reached this area. These standing wave patterns are only fixed with respect to the reflecting surfaces. This means that if the surface configurations change, the standing wave will likewise change. Thus, the construction of a new mole around the Naval Operating Base area must change the standing wave pattern along the existing shore line and within the entire area enclosed by the mole. Furthermore, the disturbance pattern in the outer harbor area adjacent to the mole will likewise be affected.

1. DIMENSIONS OF STANDING WAVE PATTERN

A standing wave pattern has a wave length. This is the distance between points of similar behaviour, for example, from one point of maximum vertical movement to the next adjacent point of similar movement. For a straight reflecting surface this dimension would be determined by the wave length of the trains and the angle between the two trains. In an actual case in which reflection occurs from several surfaces at different angles, the resulting standing wave pattern becomes very complicated. In such a case it is difficult to predict the location of maximum and minimum points of motion, i.e., the loops and nodes.

The fact that for a given set of boundaries the standing wave pattern is determined by the wave length and direction of the incoming wave train means that a given area has as many standing wave trains as there are different wave lengths that enter the area. Furthermore, since wave trains in nature are never uniform and regular, the standing wave pattern will fluctuate in cycles around the mean configuration. This fluctuation will increase with the distance from the reflecting walls.

2. EFFECT OF BOUNDARIES

The very fact that a standing wave pattern is the result of the inter-action between an original and a reflected wave train means that this standing wave pattern is very much affected by the size, shape and type of the enclosing boundaries. In particular, the standing wave pattern in a closed basin is greatly affected by the frequency of the exciting wave train. This is because in any closed basin there are opposite walls which will reflect waves back and forth. In other words, a wave is reflected from the first surface to the second, from there back to the first and again to the second, and so on. In such a system there are always certain frequencies for which the second reflection has the same phase as the first, i.e., where the second reflection amplifies the first reflection. These are the dangerous frequencies of resonance for which the wave heights are much larger for the same amount of incoming energy than for any other frequency. Thus, any closed basin with reflecting surfaces is an oscillator and especially responsive to certain frequencies.

The response of a particular basin to different exciting wave trains is governed by its own fundamental characteristics. For example, a solitary wave will travel from one end of the basin to the other and return in a given time. In making this round trip a certain amount of energy is lost. Now, if just at the instant that the wave starts on its second round trip, it is given a "push" by another wave whose energy is equal to the amount that was lost by the first wave on its round trip, the second round trip will be a duplicate of the first one. Thus, in order to keep the wave traveling back and forth continuously with no loss in amplitude, all that is necessary is to admit a small exciting wave train whose period corresponds to the time of one complete round trip of the wave in the basin and whose amplitude corresponds to the energy lost per round trip. Since for long waves this loss of energy per trip is small, a wave train of the proper frequency but of very small amplitude can maintain a continuous large amplitude oscillation in the basin. Now, if the period of the exciting wave train were to be reduced until it became one-half of this fundamental period, it would set up a double oscillation in the basin; i.e., two waves going back and forth continuously, separated by equal intervals. Since each wave is only half as long, the amount of water moved as it passes a given point is, for the same amplitude, only half as much. By the same token, it only takes half as long to pass. The result is that the induced velocity is unchanged. Hence, the total energy that must be added per unit time to compensate for that lost through damping is a constant, independent of the wave frequency. By the same reasoning, it will be seen that a similar situation will exist at each frequency which is an integral multiple of the fundamental one and that the amount of energy that must be brought in by the exciting wave train is a constant, provided that the amplitudes of the basin oscillation and the existing train remain unchanged. The time of the round trip of a single wave is the fundamental period of the basin. One-half, one-third and one-fourth of this amount correspond to the first, second, and third harmonics of the basin. If the exciting wave train has a period which is not an exact fraction of the fundamental period of the basin, i.e., if the frequency of the train is not an integral multiple of the fundamental frequency, the amplitude of the standing wave pattern which is set up in the basin will be much smaller.

3. BASIN FREQUENCIES

The above discussion shows that any new basin that is constructed will have a fundamental frequency and a series of harmonics. If an exciting wave train can enter the basin, the reaction of the basin to the wave train will depend upon whether or not the frequency of the train is the same as that of the basin or one of its harmonics.

The previous discussion considers only one path for the oscillating wave, i.e., from end to end of the basin. In a real basin, such as the one that will be formed by the mole, complications are added by the fact that there are several possible paths for the oscillating wave. The amplitude of the standing wave pattern will be reenforced in case the frequency of the exciting train corresponds to the fundamental or harmonics of any of the possible paths.

If the frequencies of two or more of these paths are whole multiples of each other, an exciting train of a frequency corresponding to the fundamental or a harmonic of the higher frequency path will excite both modes of oscillation. In such cases, the maximum amplitude of the standing wave pattern will be reduced, since the available energy is divided between two or more wave trains.

It will be seen that, due to the very nature of standing wave patterns, there is no uniformity of behaviour possible in an area in which one exists, since conditions may change quite radically within a distance of the order of magnitude of the wave length. Thus, along the Naval Operating Base shoreline, both at present and after the mole is constructed, it can be expected that the disturbance due to the normal 600 ft. wave train will not be uniform, but will vary from pier to pier and drydock to drydock.

(a) Seiches and standing waves. It is interesting to note in passing that a "seiche" is simply a standing wave generally, but not necessarily, on an enclosed body of water in which the oscillation is occurring either at the fundamental frequency of the basin or at one of the low harmonics.

4. HORIZONTAL WATER MOTIONS

So far, the discussion of the wave pattern has been confined to the characteristics of the vertical motion. It is obvious that there must be an accompanying horizontal motion to move the necessary water to form the crests and troughs. This can be seen easily in Figure 25 which shows diagrammatically a vertical cross section of a standing wave pattern. Points A_1, A_2, and A_3 are nodal points at which there are no vertical motions. The solid line represents the configuration of one wave of the standing pattern which has a wave length, L. The half-amplitude of the wave from the mean water level is H, and the period of the oscillation is T. Assume that at time, t, the wave has a configuration shown by the solid line. Then at time = t + $\frac{1}{2}$T (i.e., one-half period later) the configuration will be as shown by the dashed line. Assume also that the volume of water per foot of wave crest contained in the portion of the wave above the mean level is Q cu. ft. This is the volume of one foot of the section shown shaded in the diagram. Now consider the sections $A_1 - A_1$ and $A_2 - A_2$. In time $\frac{1}{2}$T the volume Q must flow across each section in order to permit the reversal of the positions of the crest and the trough. This is also true for section $A_3 - A_3$ and all similar sections. Therefore, the total flow in time $\frac{1}{2}T$ out of, or into any area between two nodes is 2Q, and the flow across each nodal section is Q. If the mean depth of the water is h, then the average velocity across all sections A-A must be given by the simple expression:

$$V_{A-A} = \frac{Q}{T/2h} = \frac{2Q}{Th} \tag{4}$$

If it is assumed that the wave profile is a true sine curve, then

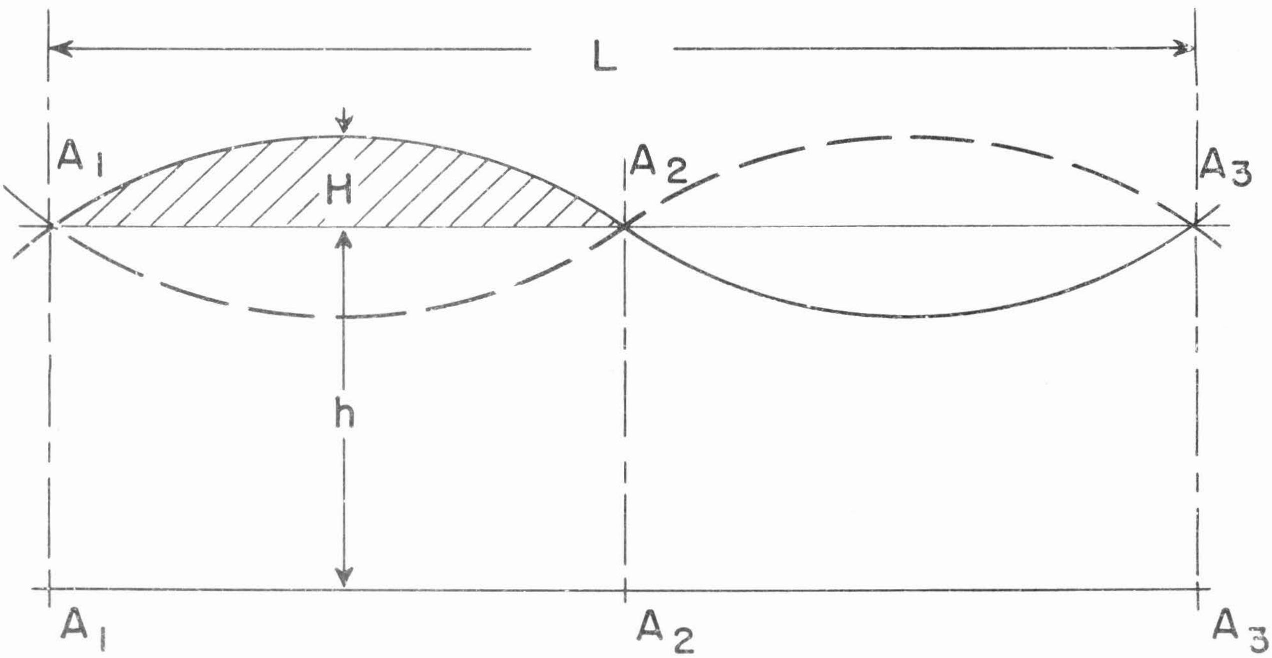


FIG. 25 DIAGRAM OF STANDING WAVE

the volume, Q, is given by the integral:

$$Q = \int_0^{L/2} H \sin\frac{2\pi}{L} x \, dx = \frac{H L}{\pi} \tag{5}$$

If this value is substituted in the expression for the average velocity, this becomes:

$$V_{A-A} = \frac{2}{\pi}\frac{LH}{Th} \tag{6}$$

The maximum velocity will be about 50% greater than this mean.

It will be observed that equation (6) contains both the wave length and the period. The former can be eliminated by introducing the relationship:

$$L = C\ T \qquad (7)$$

together with the expression previously derived for the velocity of a shallow water wave.

$$C = \sqrt{gh} \qquad (3)$$

into equation (6). The result is:

$$V_{A-A} = \frac{2}{\pi}\ H\ \sqrt{\frac{g}{h}} \qquad (8)$$

Since, at a given location, all the quantities are constant except V and H, Equation (8) indicates that the horizontal water velocity is independent of the wave length and varies directly with the wave height.

This analysis can be carried a step further by making some additional broad generalizations. Thus it may be assumed that the force tending to produce horizontal ship motion varies with the square of the water velocity. The ship acceleration will be proportional to the force acting. Therefore, the following equation can be written for the direction of horizontal motion "S" of the ship.

$$S = 1/2\ aT^2 = 1/2\ KV^2\ T^2 = K_1\ (V\ T)^2 = K_2(H\ T)^2 \qquad (9)$$

Stated in words, this means that the amplitude of the horizontal ship motion is proportional to the squares of the wave heights and the wave periods producing the motion. Here again it will be noted that much simplification has been introduced into this consideration. This is a "one-dimensional" solution. In the real case there may be two or more components of motion with different directions and different phase relations or, for that matter, they may be of even different harmonics and, therefore, different periods. One of the chief reasons for making model studies of such cases arises from this fact. The model basin acts as an integrating machine. It combines all the various components of horizontal and vertical motion and presents the result readily for measurement and recording.

E. MODELS AND MODELS SCALES

One of the first quantities to be decided upon in the planning of a model study is the model scale. In selecting this there is one general rule to be remembered. The model should be as small as is consistent with obtaining valid results. The reason for building a model at all is to obtain a solution of the problem quicker and more economically than can be done by studying it in the prototype. The larger the scale, the more expensive it will be to construct and operate the model, and the longer will be the time required for the investigation.

The basic requirement for a satisfactory model is that the behaviour of the model must be similar to that of the prototype.

both regarding motions and forces. If this rule were followed rigorously very few model studies could be carried out. The compromise which has proved satisfactory is that these conditions must be fulfilled for the main phenomena involved, but that compromises may be permitted concerning effects that are of minor importance. In many cases quite wide departures from similarity between the model and the prototype or some of the secondary phenomena have been permitted without affecting the validity of the results. This has been made possible by the applying of corrections to the results for the known deviations. These statements imply that in order to plan and carry out a successful model study it is necessary to have a clear understanding of the basic nature of the phenomena involved and the relative magnitude of their importance to the study. In other words, a model study is like any other calculating machine. The operator must understand the problem in order to set it up in the machine, and be able to interpret the reading on the dials in order to make use of the answers. These statements are somewhat at variance with the popular conception of model studies. It is often assumed that by the use of models it is possible to set in the problem and get out the answer without the necessity of having any idea of what actually goes on.

In constructing hydraulic models which have free water surfaces, one of the problems which has to be investigated is the effect of surface tension on the phenomena to be studied. In general, surface tension forces are negligible in the prototype, hence the model must never be made so small that they become of major importance. Likewise, in river and harbor problems the viscosity of the water has very little influence on the actual water movements because the dimensions are so large that even for flows at very low velocity the effects of viscosity are not appreciable. The Reynolds number is the parameter which is used to measure the relative importance of the viscous forces. For water at a normal atmospheric temperature, this reduces to the simple product of the velocity and a linear dimension such as the depth. As the product of the depth in inches and the velocity in feet per second gets below a value of about one-tenth, the flow will be laminar. If it is above this value, the flow will be turbulent. If the product exceeds unity the flow will be well into the turbulent range and viscosity will have little effect. It is obvious that laminar flow is never encountered in the prototype in river and harbor problems. It is thus desirable that for the flow in the model this product should be kept above unity.

Another factor that must be considered in deciding the scale of the model is the absolute magnitude of the measurements that must be made. In the case of a wave study such as the one under discussion the most difficult measurements to make are those of the horizontal and vertical water motions. The model scale must be chosen so that the resulting motions are large enough to be measured with satisfactory accuracy by the instruments available. In making this decision one auxiliary factor is of importance for the motion of the water surfaces involved. This is the probable magnitude of the extraneous disturbances that may occur in the model without regard to what happens in the prototype. In the present case this resolves primarily into the amount of movement that may be produced as the result of the existing light breezes in the area of the model basin. Such disturbances have an absolute magnitude independent of the model scale and thus the larger the model is made, the less their importance will be.

In many cases of river and harbor models the horizontal and vertical scales are given separate consideration. In a specific laboratory the maximum horizontal dimensions of the model are fixed by the facilities available. On the other hand, the prototype configuration determines how much area must be represented in the model if the prototype phenomena are to be reproduced. These two factors fix the maximum horizontal scale which can be used. If the consideration of the model requirements previously discussed shows that this scale is equal to or larger than that required for the operation of a satisfactory model, then the same vertical scale can be used and an undistorted model constructed. However, in problems covering a large area, such as harbor problems, the solution is seldom so simple. It is very probable that when the maximum permissible horizontal scale has been determined, it will be found that the surface tension effects will be too great, the viscosity will play too important a role, or that the movements will be so small as to offer severe difficulties in their measurement. The existence of any of these difficulties raises the question of the possibility of distorting the model. Laboratory facilities generally permit the use of a much larger vertical scale than the maximum horizontal one. If the model considerations allow this, it means that the surface tension and viscosity effects can be reduced and the magnitude of the quantities to be measured can be increased. However, such a distortion is only permissible if the resulting model behaviour will still be similar to that of the prototype after the proper allowance has been made for the difference in scales.

In general, distorted models can be used most satisfactorily in problems involving only one type of phenomenon. If the inter-action of two or more different phenomena are to be studied simultaneously, it will usually be found that the distortion affects each phenomenon differently and thus their interaction in the model may have little similarity with that in the prototype. The present harbor study is, as has been stated, basically one of wave mechanics only and therefore model distortion is not automatically ruled out.

In Section IV-B on wave types, it was pointed out that there are two limiting types of waves - deep water waves and shallow water waves - and that the present study involves only shallow water waves. A moment's consideration will show that a distortion in the direction of the increase of the vertical scale will increase the ratio of the depth to the wave length, which means that the waves will behave more nearly like deep water waves. Now the depth in the harbor varies considerably, hence the velocity of the shallow water prototype waves will vary from place to place and this variation will affect the wave pattern. If a model distortion is used that is great enough to result in turning the model waves into deep water waves, this variation in velocity will not occur and consequently the standing wave pattern will be incorrect.

A careful consideration of all the factors involved in the model performance resulted in the decision to use different distortions for the two sets of models. In the preliminary small scale studies it was decided to use a rather large distortion because it was felt that the advantages of greater vertical motion and ease of operation would overbalance the disadvantage that the waves would all be of the deep water type instead of the shallow water ones that exist in the prototype. This latter factor was not felt to be particularly serious for the preliminary studies because it was not planned to make any measurements of the actual standing wave pattern. However, the same chain of reasoning pointed to the desirability of using a very small distortion for the final studies of the larger scale model so as to make significant measurements of standing wave patterns, both in their configuration and amplitude. In Section IV-B it was shown that the largest ratio of depth to wave length that will be encountered in the prototype is about 1 to 12. Figure 24 shows that such waves differ from the performance of a solitary or true shallow water wave by only about 5%. If the vertical scale is increased to twice that of the horizontal, this ratio of depth to wave length will increase 1 to 6. Such waves differ by about 15% from the true shallow water performance, although they are still very sensitive to changes in depth. It was felt that this deviation is as large as should be permitted in the study of the final models.

V. RIPPLE TANK

In Section III-A in discussing the plan of the model study, the use of the ripple tank as a pilot model was outlined.

A. OUTLINE OF EQUIPMENT

A general view of this equipment is shown in Figure 26 and a line diagram of the optical method of observation is seen in Figure 27. A more complete description of the equipment will be

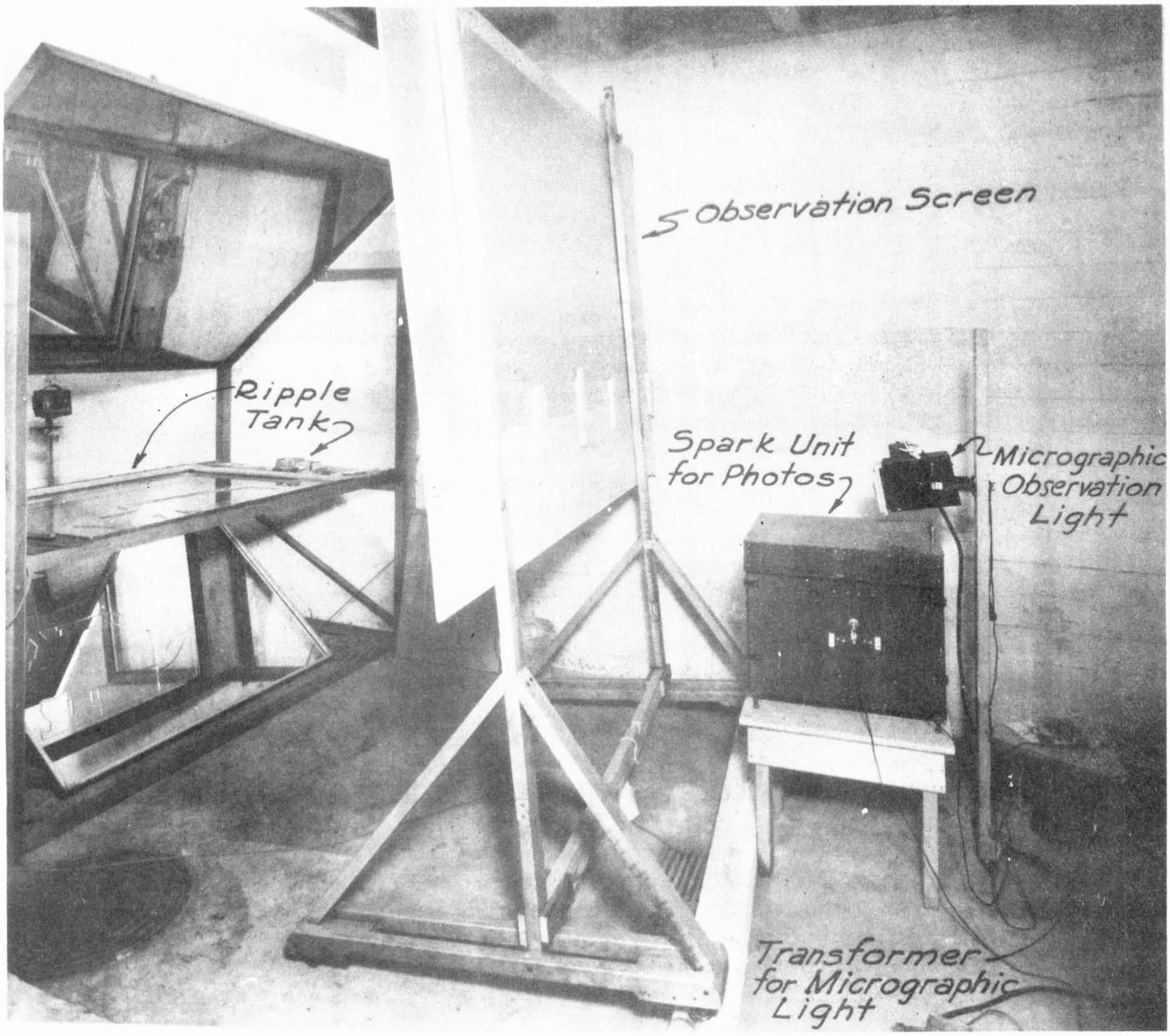


FIG. 26 GENERAL VIEW OF RIPPLE TANK AND OPTICAL EQUIPMENT

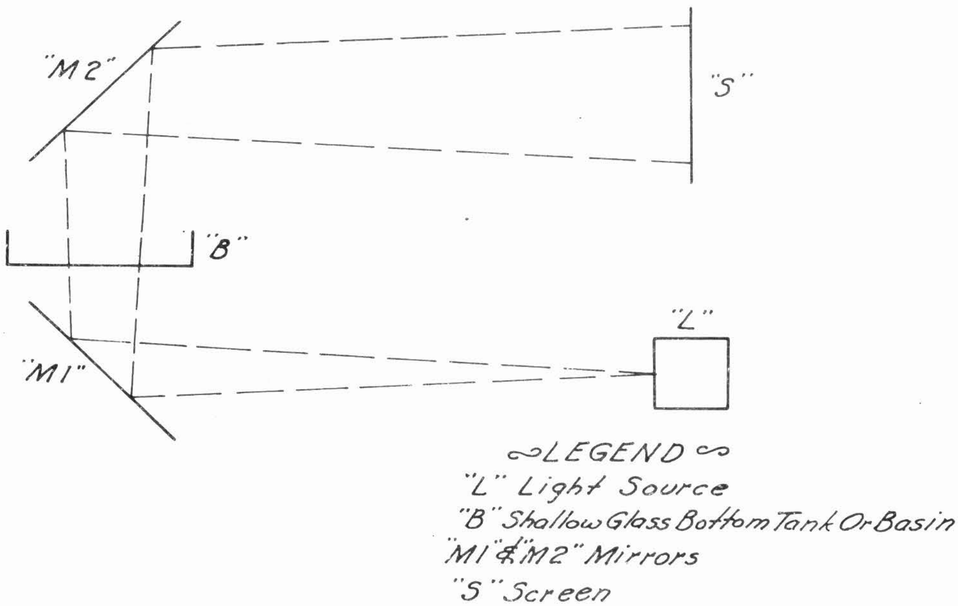


FIG. 27 SCHEMATIC DIAGRAM OF RIPPLE TANK

found in Appendix I. Basically, this equipment consists of a shallow glass-bottomed model basin B, in which the shorelines and other features of the model to be studied are constructed. A cone of light from the point source, L, is projected on the mirror, M1, up through the model basin to the mirror, M2, and thence to the observing screen, S. If the water surface is smooth the area on the screen inside the model boundaries will be evenly illuminated. However, disturbances on the surface, such as waves, refract the light so that it does not reach the screen and thus they appear as distinct dark lines that can be observed easily and photographed. In these models no attempt is made to represent the vertical dimensions to scale. Shoreline and other construction features such as moles and breakwaters were cut to a true horizontal scale from one-half inch thick lucite sheet. The horizontal contours were cut with vertical faces. In operation the basin was filled with water to a depth of about 1/4 inch. Two complete models were constructed. The first one covered the entire coast from Point Fermin to Sunset Beach and included the outer breakwater and a portion of the ocean to seaward. The second one covered only the area in the immediate vicinity of the Naval Operating Base and the proposed mole. Figures 28 and 29 show the first and second model, respectively. It will be noted that in the area covered they correspond to the preliminary and final models in the main basin.

B. RESULTS FROM FIRST MODEL

The first objective of the study was to determine the general wave pattern to be expected in the outer harbor as a result

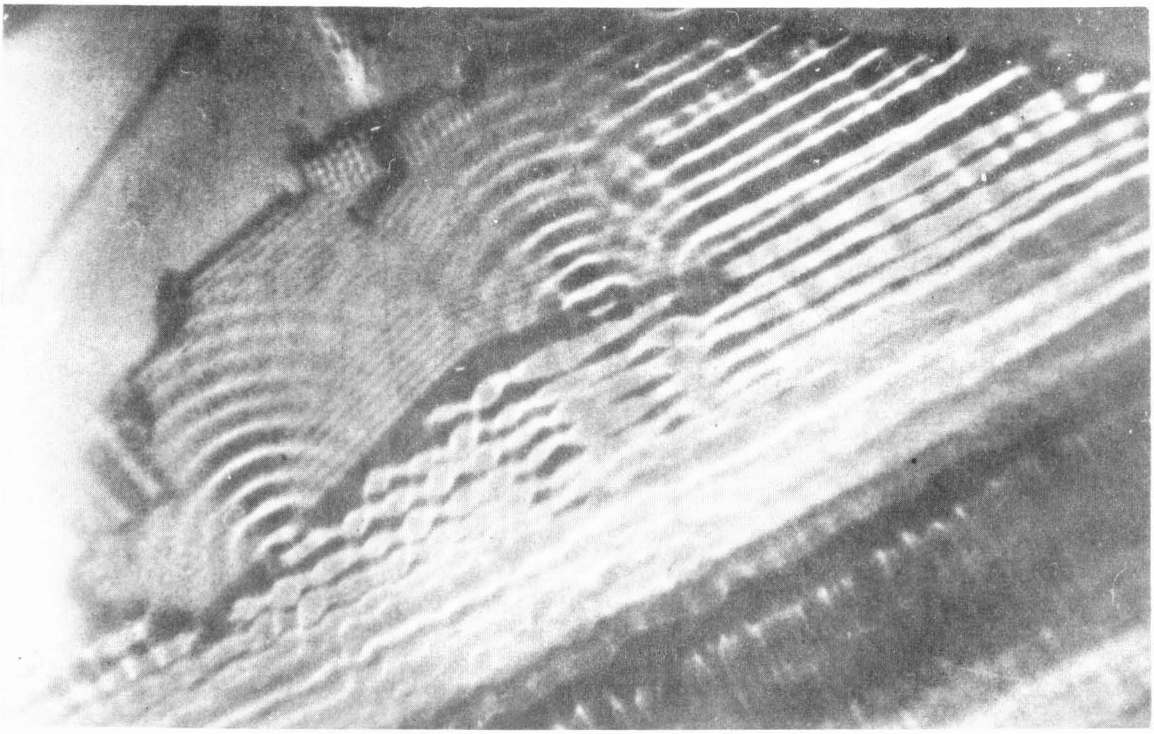


FIG. 28 RIPPLE TANK, MODEL NO. 1 COVERING AREA FROM POINT FERMIN TO SUNSET BEACH

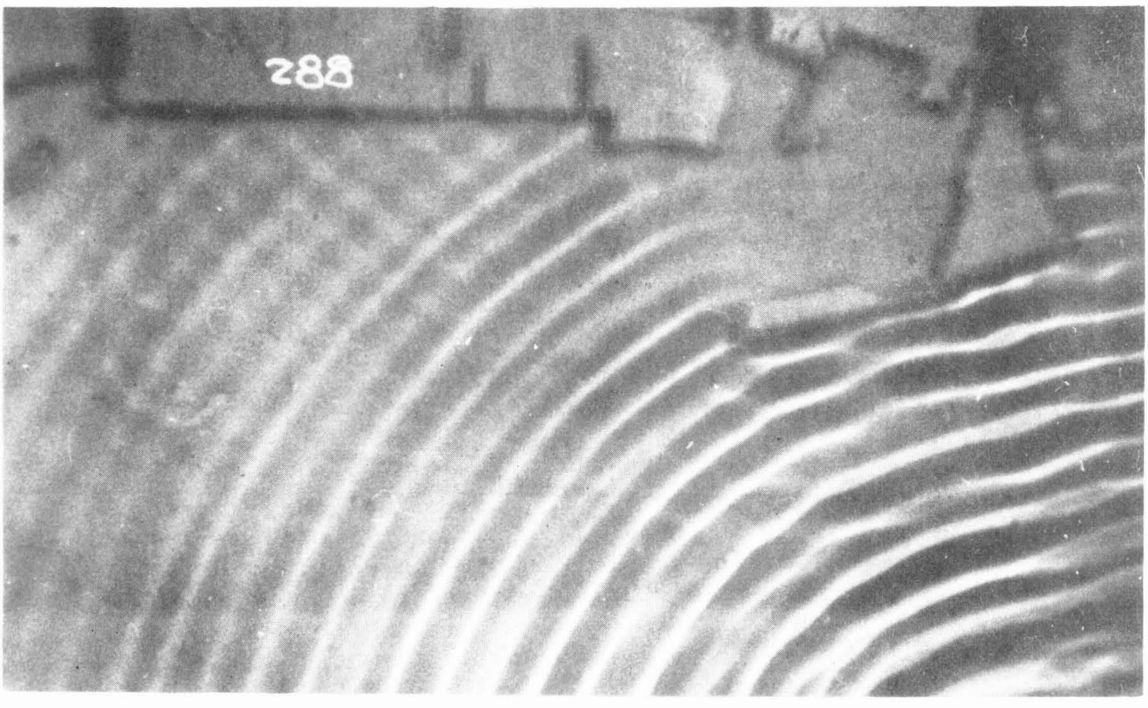
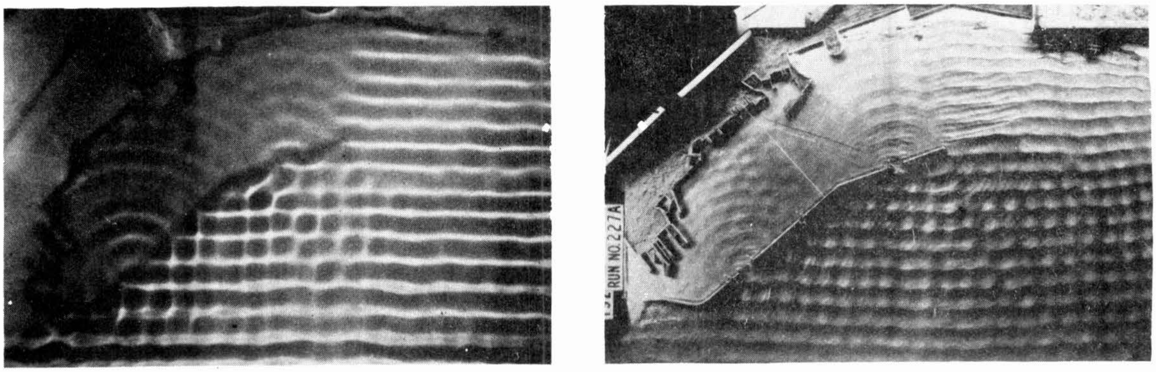


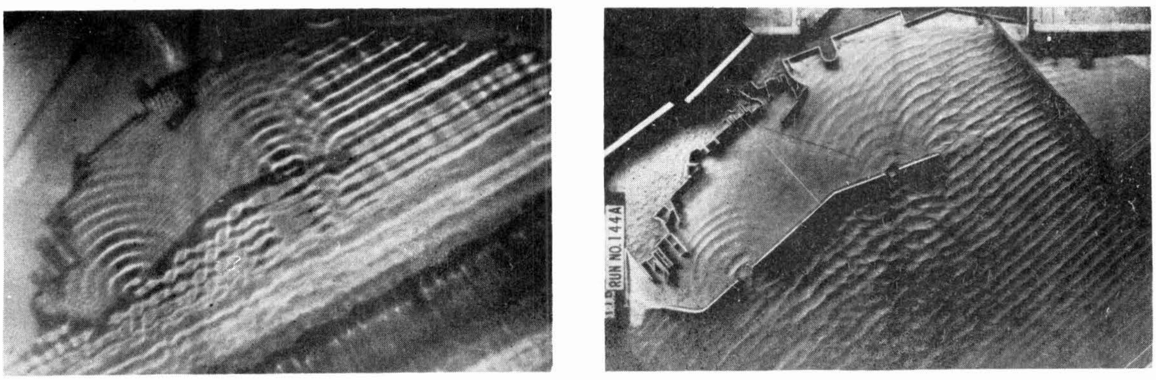
FIG. 29 RIPPLE TANK, MODEL NO. 2 COVERING DRY DOCK AND LONG BEACH HARBOR AREA

of normal and storm waves in the open ocean. Figures 30 and 31 show ripple tank pictures taken with waves coming from both the normal and the storm directions.



RIPPLE TANK MODEL BASIN

FIG. 30 MODEL NO. 1 WITH WAVES FROM NORMAL DIRECTION



RIPPLE TANK MODEL BASIN

FIG. 31 MODEL NO. 1 WITH WAVES FROM STORM DIRECTION

These figures also show the wave patterns obtained later in the large basin and demonstrate that a good qualitative picture was furnished by the ripple tank. These results showed clearly that with normal waves the west gate was the principal source of disturbance, that the east gate contributed less and that the open end contributed very little. It was recognized that even this contribution of the open end was more than would be expected in the prototype because the studies in the large basin showed that a sloping beach east of the Los Angeles River would probably damp out most of the energy that was seen to go around the end of the breakwater in the ripple tank.

All the studies in the ripple tank showed that, as far as the short period waves were concerned, the portions of the wave trains that came through the gates were sufficient to cause considerable disturbance in the Naval Operating Base area, and, for that matter, in the entire outer harbor. Consequently, an exploratory study was made to see how much modification would be required at the gates to decrease appreciably the amount of wave energy coming in through them. Figures 32 and 33 show the results of this study. In most of the gate configurations proposed an attempt was made to keep the navigation entrance as unrestricted as possible. The results of these studies indicate that there is a possibility of reducing appreciably the magnitude of the disturbance due to these short (600 ft.) waves that come through the gate. However, it was quite clear that the scale of the changes, i.e., the length of spurs, etc., that would be necessary to accomplish this would have to be reasonably large as compared to the wave length. In other words, major modifications of the gate openings would be required to produce any satisfactory reduction of the wave disturbance coming through them.

C. RESULTS FROM SECOND MODEL

The second, or larger scale, model was used for the study of possible changes in the configuration, gate location, opening and other construction features of the proposed mole. Figure 34 represents conditions in the basin with and without the mole. Figure 35 shows the comparative effect of two different gate openings, and Figures 36 and 37 present a more detailed study of gate protective constructions to see whether or not it would be feasible to produce a further reduction of energy at the mole gates by using constructions similar to those tried at the breakwater gates in the first model. As the photographs show, it is possible to reduce the disturbance in this manner, but only by sacrificing ease and flexibility of navigation. Also, the cost of construction undoubtedly would be increased.

During the course of the study in the large model basin an interesting phenomenon was observed. A dredged channel exists from the proposed mole opening to the east gate of the breakwater. It was observed that in the water over this dredged channel the amplitude of the waves in the train that came from the east gate was lower than it was on either side. This phenomenon was explored further in the ripple tank with the results as shown in Figure 38. It will be seen that the ripple tank studies confirmed the observation and at the same time threw more light on the nature of the phenomenon. This damping effect of the waves in the deep channel may be explained from at least two points of view. It should be emphasized that these explanations are not conflicting, but simply different ways of describing the same thing. The first explanation is very simple. The disturbance is a wave train of shallow water waves and, hence, the velocity of each wave is determined by the depth. That portion of the wave crest which lies in the deep water channel will move faster than the segments of the same wave crest lying on each side of the channel where the water is shallower. Hence, the portion of the crest in the deep channel tends to pull ahead. The relative displacement of the trough and crest between the different segments

STORM WAVES FROM SOUTHEAST — GATE CONFIGURATION — NORMAL WAVES FROM SOUTH AND SOUTHWEST

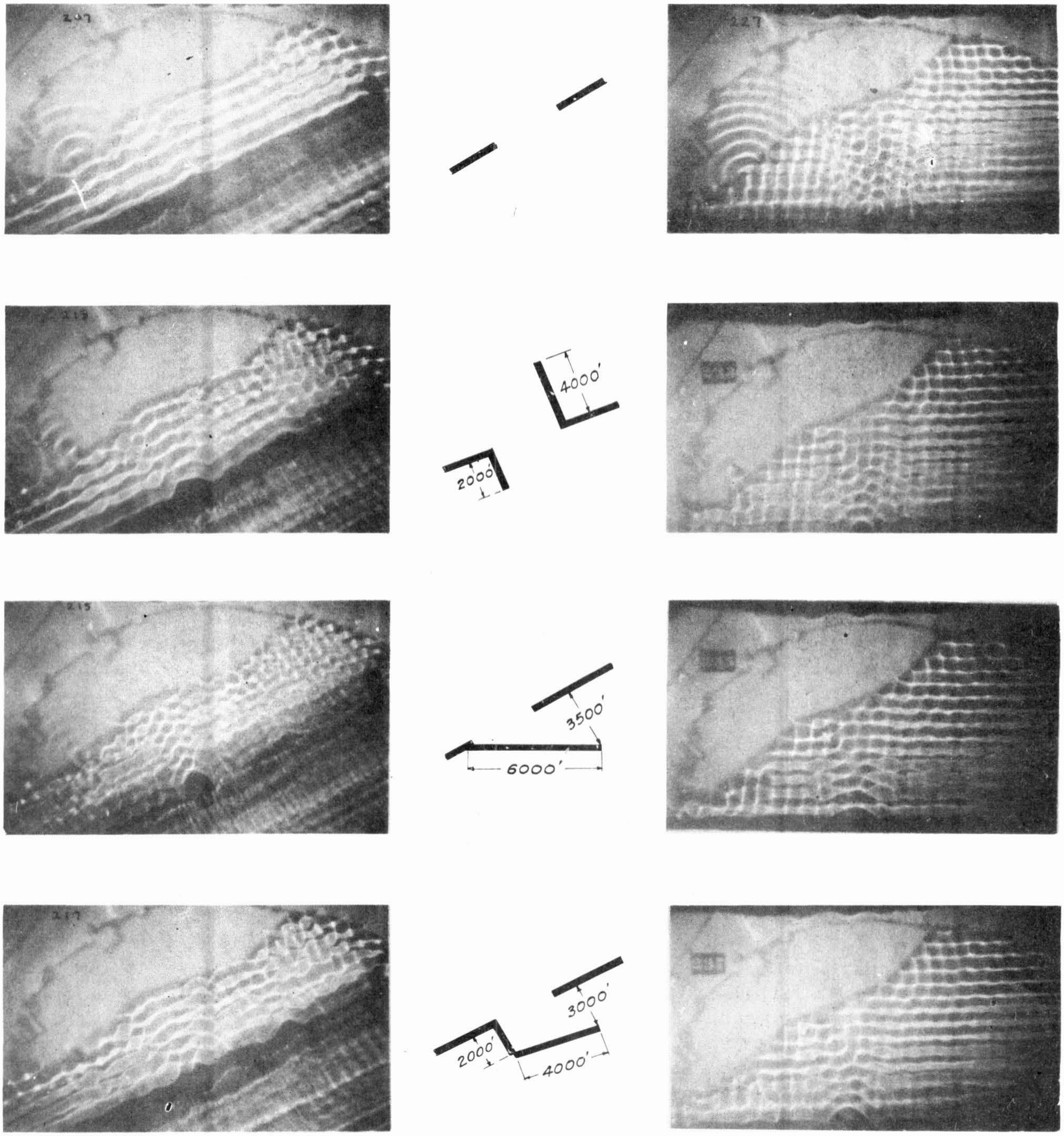


FIG. 32 RIPPLE TANK STUDY - WEST GATE MODIFICATIONS WITH EAST GATE AND EAST END OF BREAKWATER CLOSED

STORM WAVES
FROM SOUTHEAST

GATE
CONFIGURATION

NORMAL WAVES
FROM SOUTH AND SOUTHWEST

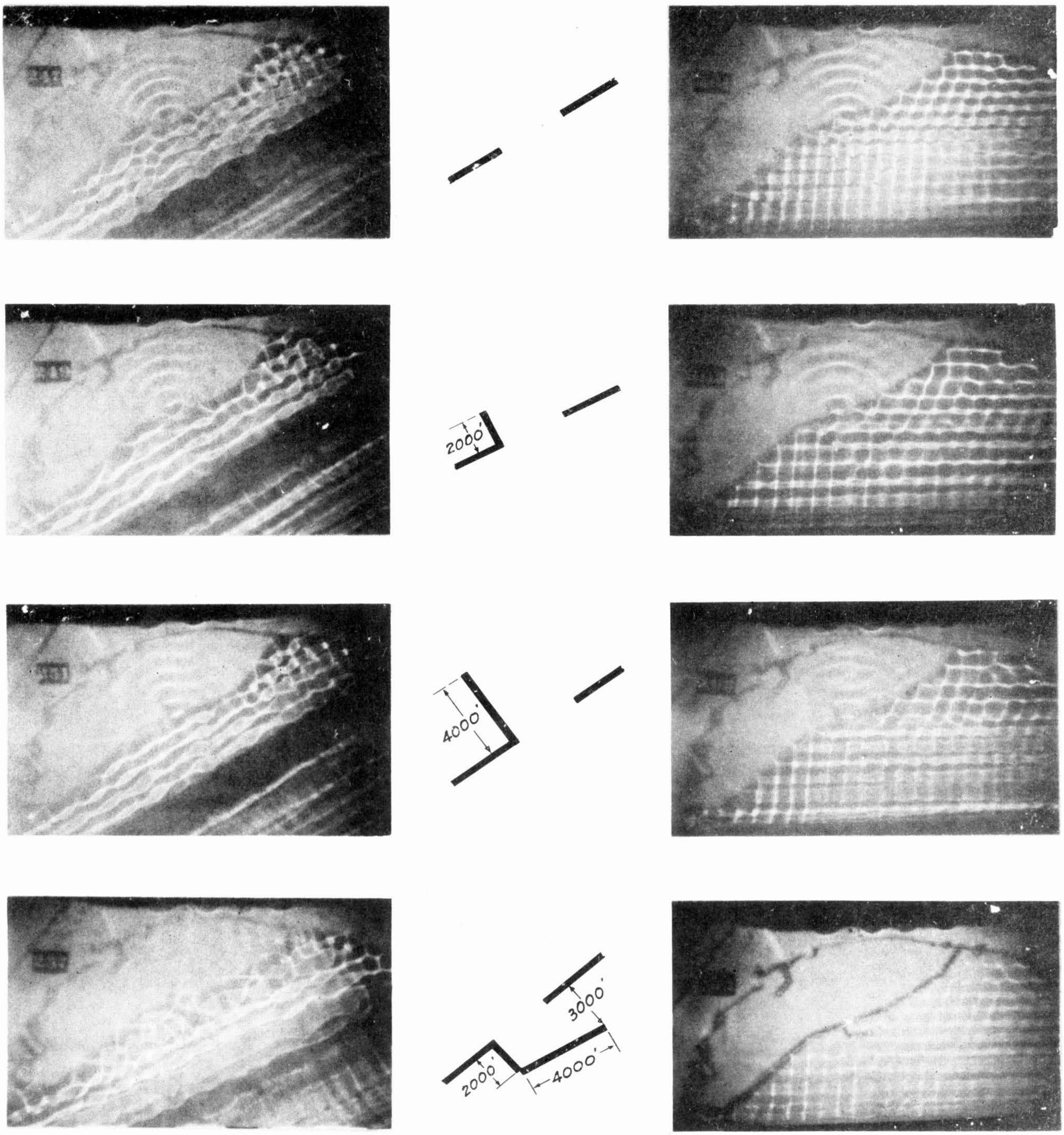
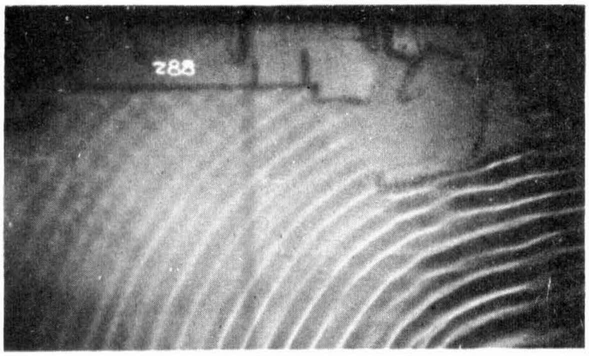
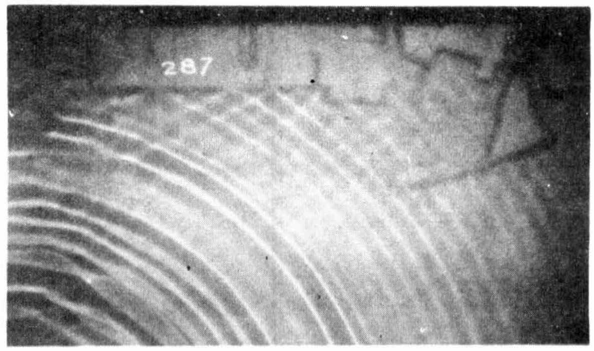


FIG. 33 RIPPLE TANK STUDY - EAST GATE MODIFICATIONS WITH WEST GATE AND EAST END OF BREAKWATER CLOSED

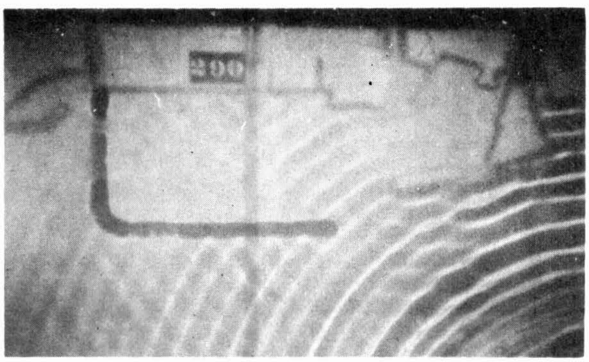
WAVES FROM EAST GATE

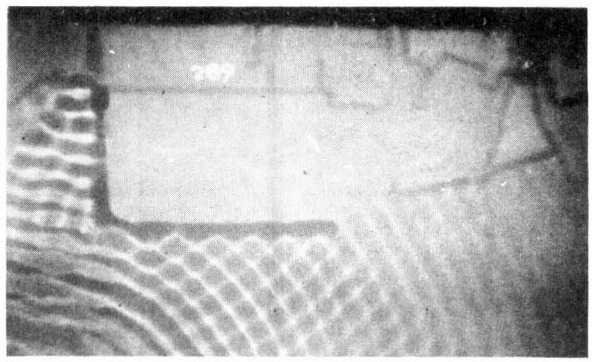
WAVES FROM WEST GATE





BASIN WITHOUT MOLE



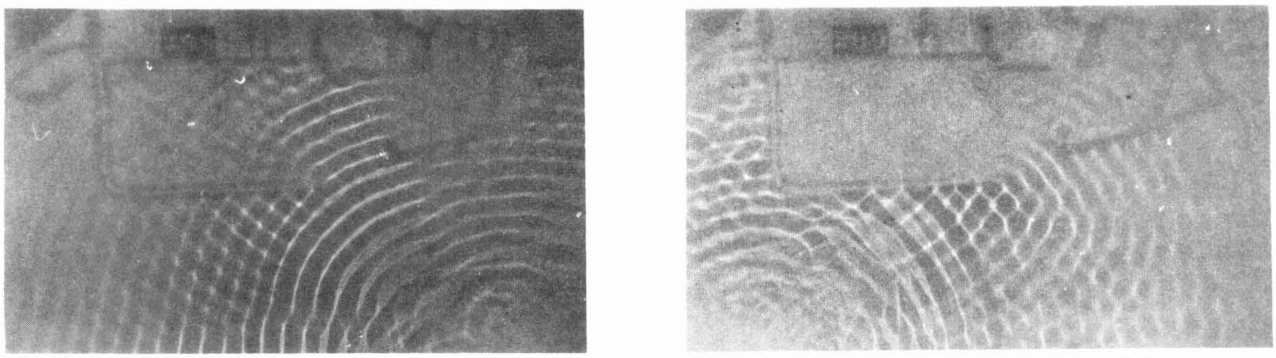


BASIN WITH MOLE

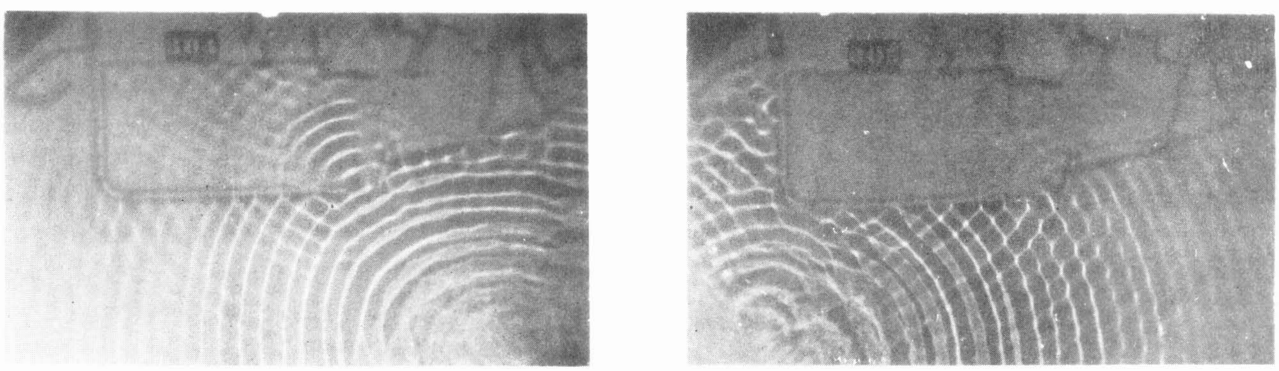
FIG. 34 RIPPLE TANK STUDY – INFLUENCE OF STANDARD MOLE ON WAVES REACHING THE DRYDOCK AREA

WAVES FROM EAST GATE

WAVES FROM WEST GATE



MOLE GATE OPENING 2070 FT.



MOLE GATE OPENING 750 FT.

FIG. 35 RIPPLE TANK STUDY - INFLUENCE OF THE WIDTH OF THE MOLE GATE

WAVES FROM EAST GATE

WAVES FROM WEST GATE

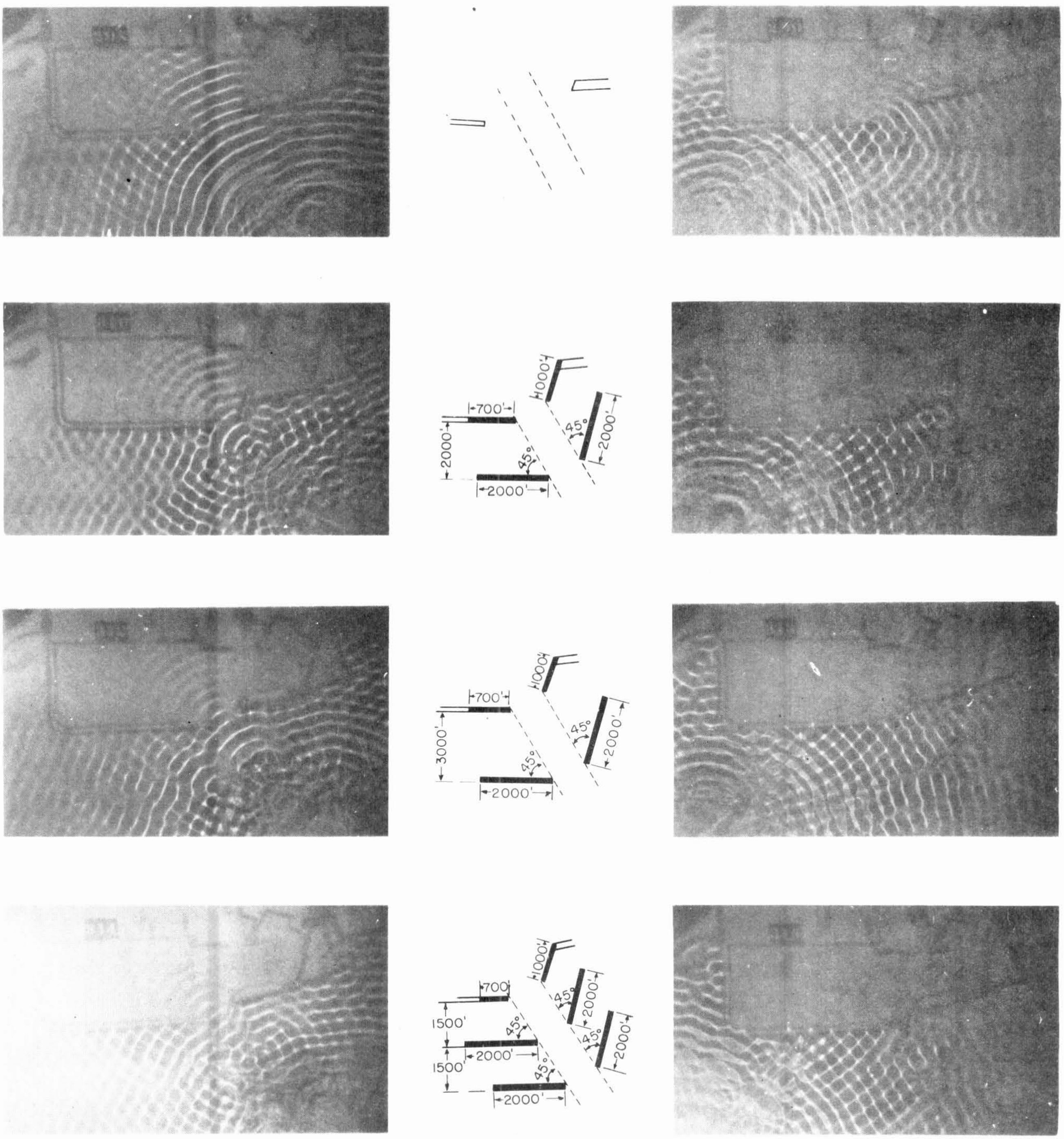


FIG. 36 RIPPLE TANK STUDY - PROTECTIVE CONSTRUCTIONS AT THE MOLE GATE (BAFFLES)

WAVES FROM EAST GATE

WAVES FROM WEST GATE

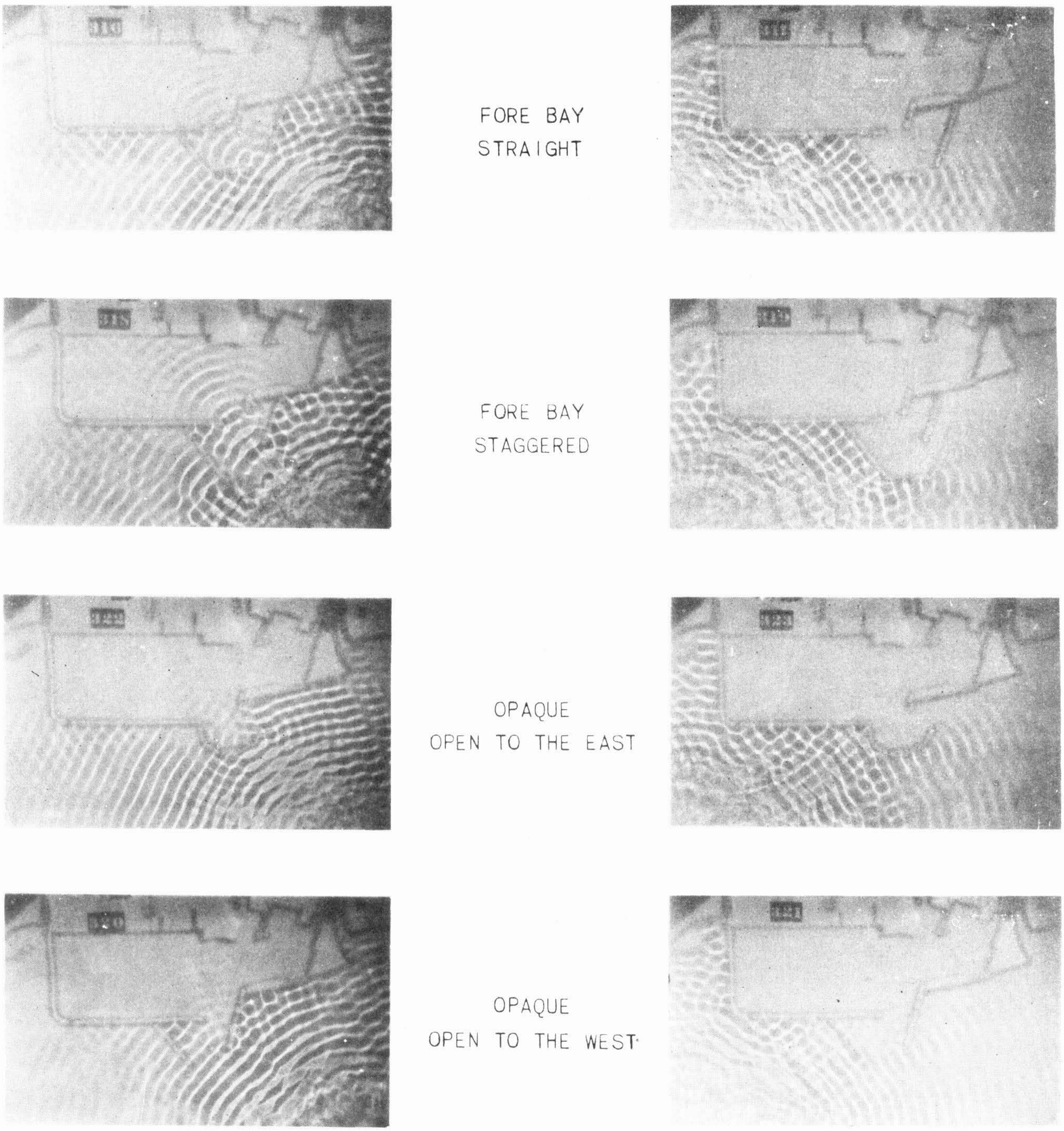
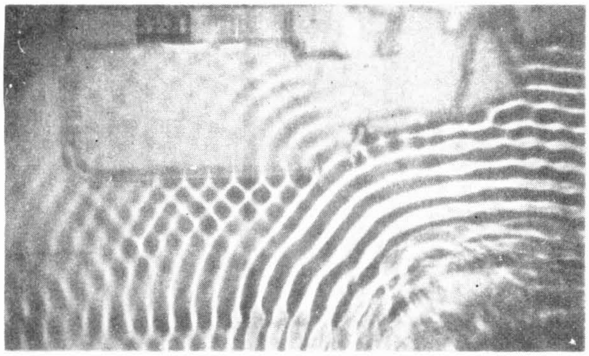
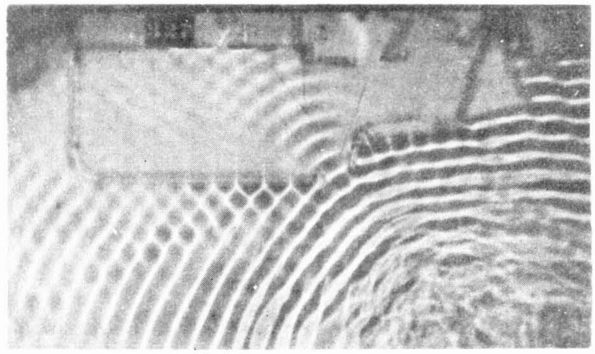


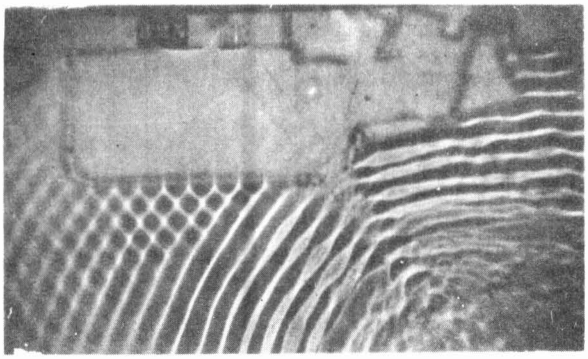
FIG. 37 RIPPLE TANK STUDY - PROTECTIVE CONSTRUCTIONS AT THE MOLE GATE (FORE BAYS)



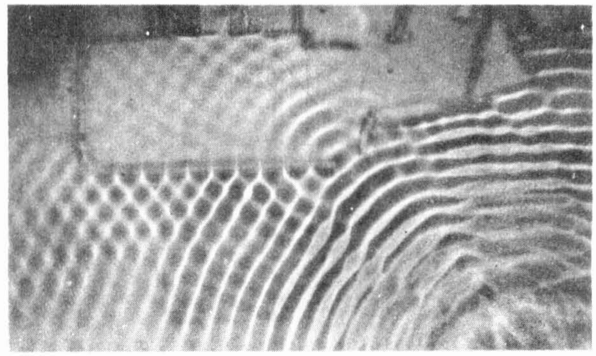
THE WHOLE HARBOR DREDGED TO CHANNEL DEPTH



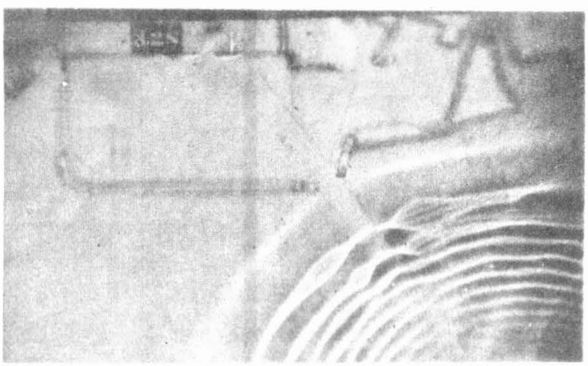
ORIGINAL HARBOR DEPTH WITHOUT CHANNEL



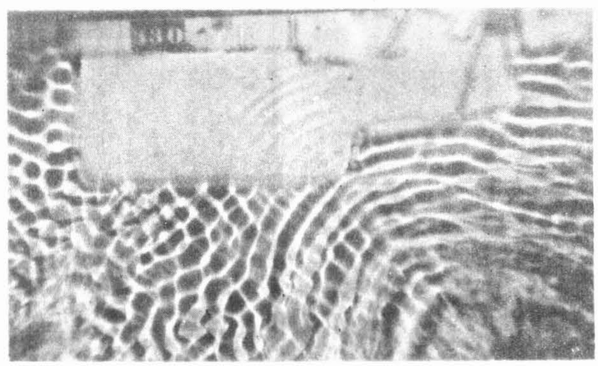
CHANNEL DREDGED 1 - WAVE LENGTH WIDE 8-10 WAVE LENGTHS LONG



CHANNEL DREDGED 3-WAVE LENGTHS WIDE 8-10 WAVE LENGTHS LONG



SINGLE WAVE MOVING ALONG EXISTING CHANNEL



EXTREMELY STRONG WAVES ENTERING HARBOR THROUGH BOTH GATES WITH EXISTING CHANNEL

FIG. 38 RIPPLE TANK STUDY - INFLUENCE OF DREDGED CHANNEL ON WAVES ENTERING THE HARBOR THROUGH EAST GATE of the wave allows the crests of the deep water segments to spread into the troughs of the adjacent shallow water ones, and conversely, the crests of the shallow water segments spread into the troughs of the deep water group, although the mechanism involved is quite different in the two cases. Since the length of the deep water segment is very short, i.e., of the same order of magnitude as the wave length, and since the lengths of the adjoining shallow water segments are very long, the visible result is that the deep water segment of the train is damped considerably, the damping increasing with the distance from the breakwater gate.

A more analytical explanation is as follows:

The equation for wave propagation in two dimensions *when the vertical acceleration is neglected* is given in Lamb's Hydrodynamics (4), Page 291. For a sheet of water of variable depth h, the surface elevation satisfies the equation

$$\frac{\partial^2 \zeta}{\partial t^2} = g \left\{ \frac{\partial}{\partial x} \left(h \frac{\partial \zeta}{\partial x} \right) + \frac{\partial}{\partial y} \left(h \frac{\partial \zeta}{\partial y} \right) \right\} \qquad (9)$$

where

x and y are the horizontal coordinates

g is the acceleration of gravity

t is time

For the case of uniform depth Equation (9) reduced to the simple wave equation

$$\frac{\partial^2 \zeta}{\partial t^2} = C^2 \left(\frac{\partial^2 \zeta}{\partial x^2} + \frac{\partial^2 \zeta}{\partial y^2} \right) \qquad (10)$$

where C is the wave velocity or

$$C^2 = gh \qquad (11)$$

The problems of water waves in two dimensions are thus analogous to those of the propagation of sound in two dimensions and to those of the vibration of a stretched membrane. When the rotation of the earth is taken into consideration, the theory is given by Lamb (4) on Page 318, the equation being now

$$\frac{\partial u}{\partial t} - 2\,wv = -g \frac{\partial \zeta}{\partial x} \qquad (12)$$

$$\frac{\partial v}{\partial t} + 2\,wu = -g \frac{\partial \zeta}{\partial y} \qquad (13)$$

$$\frac{\partial \zeta}{\partial t} = -\frac{\partial}{\partial x}(hu) - \frac{\partial}{\partial y}(hv) \qquad (14)$$

where u and v are the velocities in the horizontal direction x and y respectively relative to the axis, and w is the angular velocity, the rotation being about an axis normal to the xy plane. A more general theory of water waves in which viscosity is taken into consideration has been given by Arakawa (5), but these effects and the effect of the earth's rotations are too small for our present problem, so acoustical, optical, seismological, and electrical analogues can be safely used.

The transfer of the energy of progressive waves from a channel to the adjacent shallow water may be just one example of a peculiarity in wave propagation which has been noticed in geophysical prospecting by the explosion method, and is of importance in connection with the explanation of anomalous propagation of sound waves, electromagnetic waves, and waves of light. In geophysical prospecting, for instance, it is found that a ray from a source seems to strike the boundary between two media at the critical angle for total reflection, and then to run for a time in the faster medium parallel to the boundary, finally reentering the slower medium at the critical angle just as if it were totally reflected but at a different place. The phenomenon seems to be connected mathematically with the fact that the primary waves from a point source cannot be analysed completely into *homogeneous* plane waves for which the laws of reflection and refraction are well known. The analysis of Weyl (6) indicates that, in addition to the homogeneous plane waves, some nonhomogeneous plane waves must be considered, and for these the extensions of the laws of Fresnel must be used.

In 1938 O. V. Schmidt tested experimentally the behavior of a spherical pulse produced near the plane dividing a salt solution (velocity of sound 1600 meters a second) from xylol (velocity of sound 1175 meters a second). With the aid of the multiple spark method of Cranz and Schardin he got Schlierenphotographs of the different waves. They showed clearly (see diagram below) that, in addition to the ordinary incident reflected transmitted waves, there is a fourth wave which has a conical wave front (in the three-dimensional propagation) tangential to the spherical wave in the slower medium and having a trace on the boundary which travels with the speed of propagation in the faster medium. This fourth wave is thought to be analogous to the oblique part of the wave front which causes the transfer of energy from the deep channel to the shallow water.

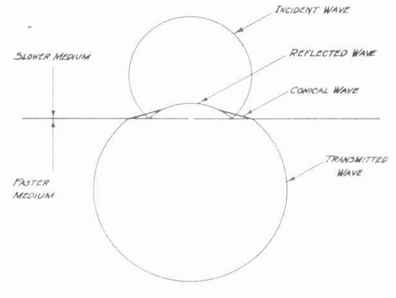


DIAGRAM SHOWING THE PRESENCE OF A CONICAL WAVE IN THE TRANSMISSION OF A PULSE FROM ONE MEDIUM TO ANOTHER

Since the appearance of Schmidt's work there has been much mathematical discussion of the acoustical, geophysical, and electromagnetic problems. Joos and Teltow (8) showed that the conical wave front can be derived from Sommerfeld's theory of the spreading of electromagnetic waves along the surface of the earth, but their work was limited to the case in which the source was very close to the boundary. A more complete analysis of the electromagnetic problem for a source away from the boundary has been made by Kruger (9) and other writers. The acoustical and geophysical problems have been discussed very fully by T. Sakai and S. Syono (11). They have even discussed the acoustical problem for the case of an atmosphere divided into two parts when reflection at the surface of the earth is also taken into consideration. This problem, which is most closely allied to the channel problem, provides the solution of a point source for an infinite slab of material between two unlimited spaces filled with the same different material. A transition to the two-dimensional problem of wave propagation in a channel when the waves arise from a point source can be made by integration. A further integration is then necessary to give the solution for the case in which point sources are distributed along a line perpendicular to the direction of mean propagation of the waves.

Geometrically the phenomenon of the transfer of energy from the channel can be regarded as a consequence of the Principle of Huygens or of the Principle of the Continuity of the Wave Front (13). There must, in fact, be an oblique portion of the wave front between two parallel straight portions and this oblique portion indicates that there is a flow of energy from the deep channel into the shallow water, but the geometrical argument gives no indication of the amount of this flow or of its dependence on the wave length of the waves and the angle of divergence (if any) of the channel. Much may depend also on the ratio of the width of the channel to the wave length. Some experiments were made with divergent channels with good results in some cases. Experiments also indicated that for a given wave length there was an important influence of the width of the channel, so there are many matters that need full investigation.

When the slower medium is imagined to be continued beyond the surface of discontinuity of the material, the conical wave front may be imagined as coming from a cone whose vertex is the image of the point source. It may also be regarded as coming from a line of sources extending from this image to infinity, each source having a proper intensity and phase. Coulomb (14) considered the last type of representation in the study of earthquake waves. Since the appearance of his paper, much work has been done on line integrals involving the elementary wave potential (15). Some tables have now been prepared by men working on different projects (18) and it is hoped that numerical solutions of some of these difficult mathematical problems will eventually be obtained.

VI. PRELIMINARY STUDIES IN MODEL BASIN

The objective of this group of studies was outlined in paragraph B of Section III on the Plan of the Model Study.

A. OUTLINE OF EQUIPMENT

A general view of the model basin with the preliminary model in place is shown in Figure 39. It will be seen that the coast line from Point Fermin to Sunset Beach is included. The entire

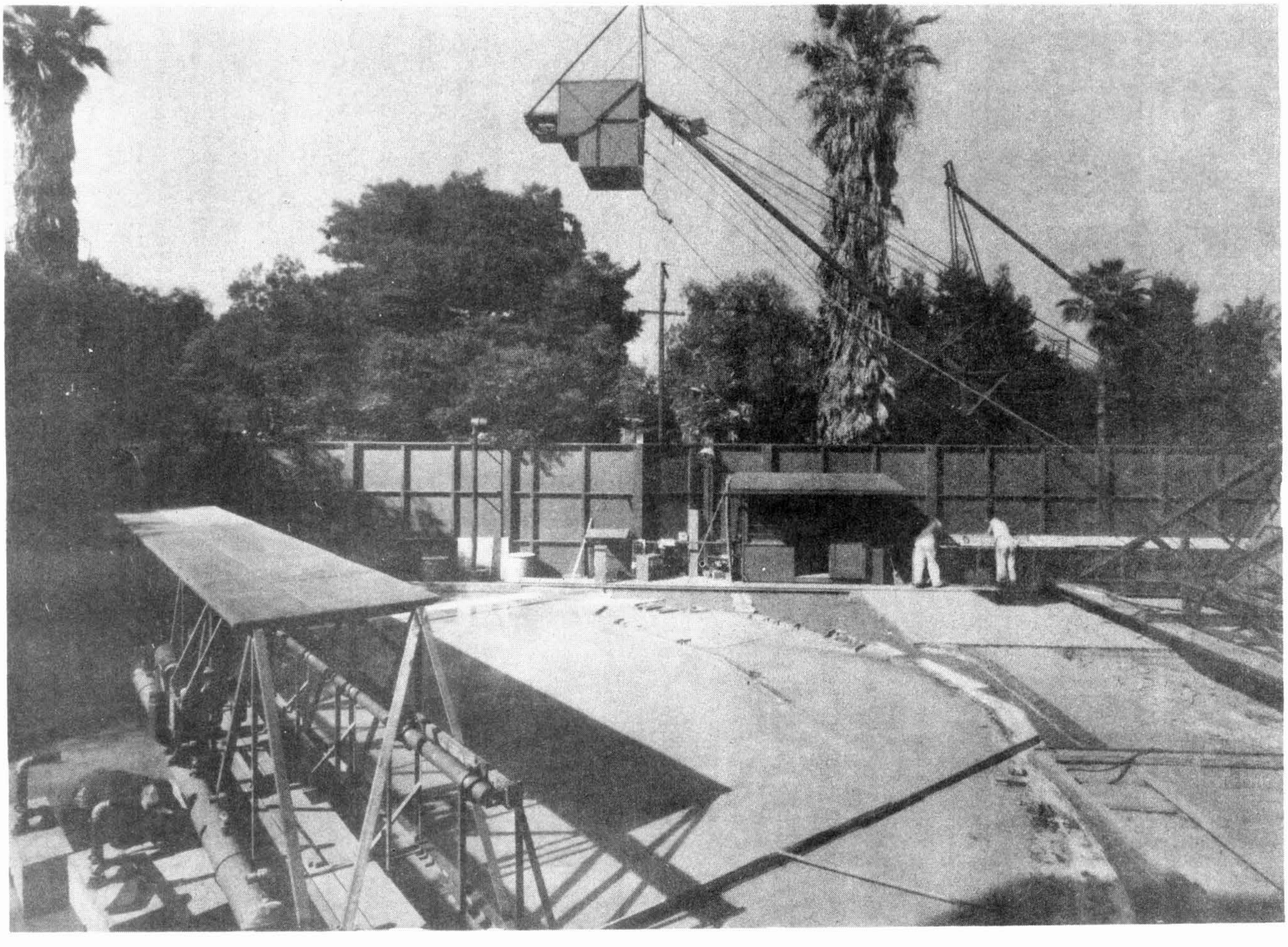


FIG. 39 GENERAL VIEW OF MODEL BASIN WITH FIRST MODEL IN PLACE

length of the breakwater is represented, together with six miles of the open ocean lying southerly of the breakwater. The horizontal scale used was 1 to 1800, whereas the vertical scale was 1 to 300. This means that the distortion in the model was 1 to 6.

These scales, and their ratio, conform to the considerations prescribed in Section IV-E, Page 44. When the study was first started the then existing wave machine was used for producing the wave trains in the open ocean. This machine proved to be unsuited to the high frequencies demanded for this study and, therefore, was replaced by the wave machine shown in Figure 40. A more complete description of the model basin and its equipment will be found in Appendix II.

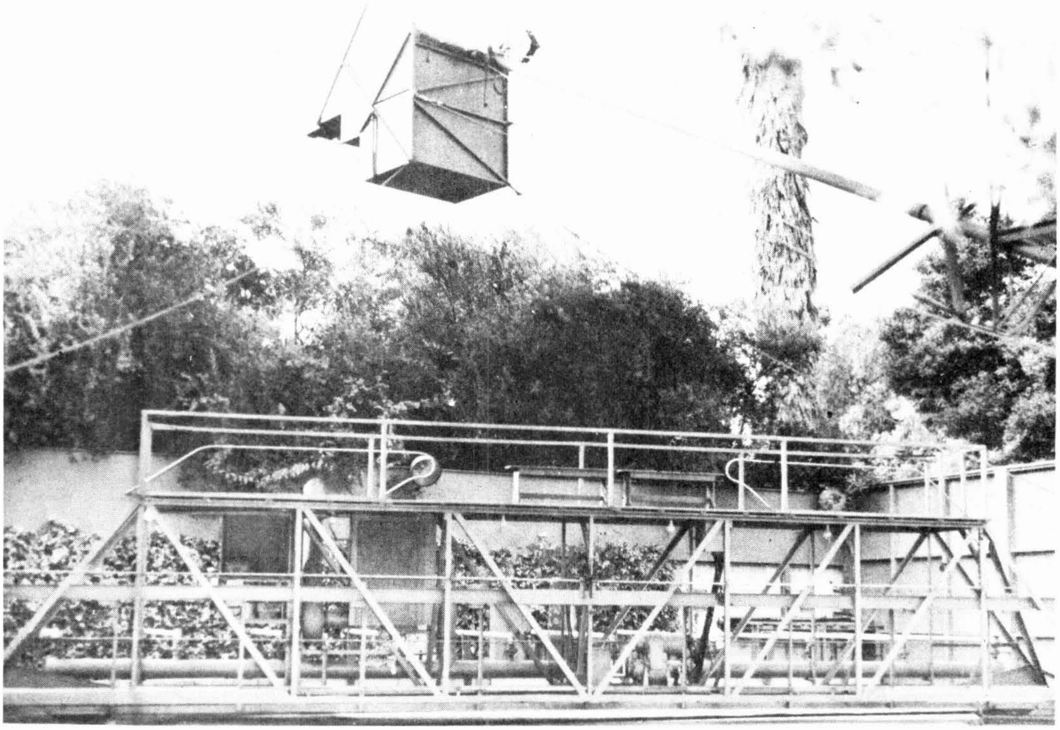


FIG. 40 WAVE MACHINE

B. SUMMARY OF RESULTS

1. GENERAL WAVE PATTERN

The first objective of this study was to investigate the general wave pattern in the outer harbor and in particular in the vicinity of the Naval Operating Base area. Figure 41 shows the wave pattern which results from the wave train approaching the breakwater from the southwest. This is the normal direction of the waves. This photograph should be compared with Figure 30 from the ripple tank and Figures 42, 43 and 44, which are aerial photographs of the breakwater gates when the wave train in the ocean is coming from approximately the same direction. It will be observed that a striking similarity between the patterns seen in the models and in the prototype offers qualitative confirmation of the conclusion reached from the study of the field data that the prototype breakwater could be considered nearly impervious to the trains of short wave length and that the gate openings would be the principal sources of disturbance from waves having periods of the order of fifteen seconds.

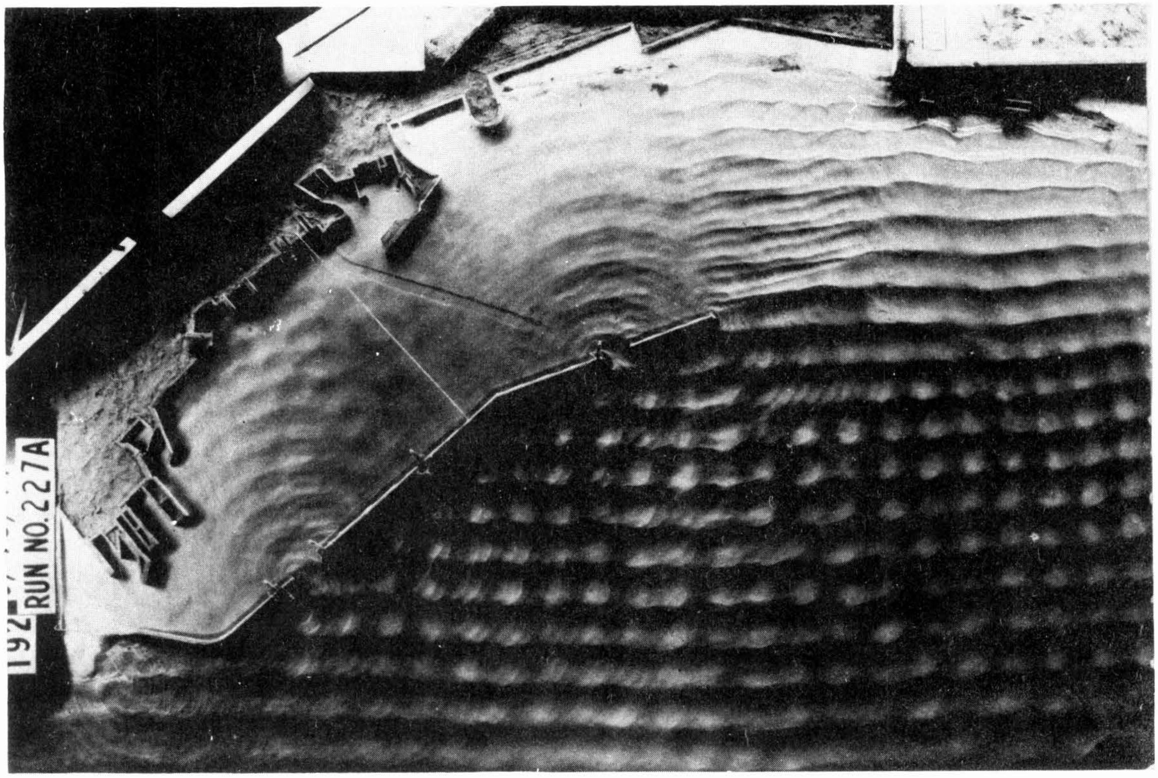


FIG. 41 MODEL NO. 1 WITH WAVES FROM THE SOUTHWEST

2. RELATIVE IMPORTANCE OF OPENINGS IN BREAKWATER

The second step in the study was to investigate the relative importance of the west gate, east gate, and the open east end of the breakwater as sources of disturbance in the Naval Operating Base area. It developed that an integral part of this investigation was the determination of the effect of the change in wave direction in the open ocean. The criterion used to evaluate both of these effects was the magnitude and configuration of the disturbance pattern in the critical area of the Naval Operating Base.

(a) Action of openings with normal wave trains. Figure 45 shows the wave pattern resulting from the normal wave train approaching from the southwest. For comparative purposes, comparable photographs are shown both for the model basin and for the ripple tank. The upper set shows the pattern with all openings functioning normally. The second set shows conditions with the west gate open and the east gate closed. In the third set the east gate is the only one open; and in the last picture the open east end is the sole communication with the ocean. It will be observed that for this normal wave direction, the greater part of the disturbance that reaches the Naval Operating Base shoreline comes

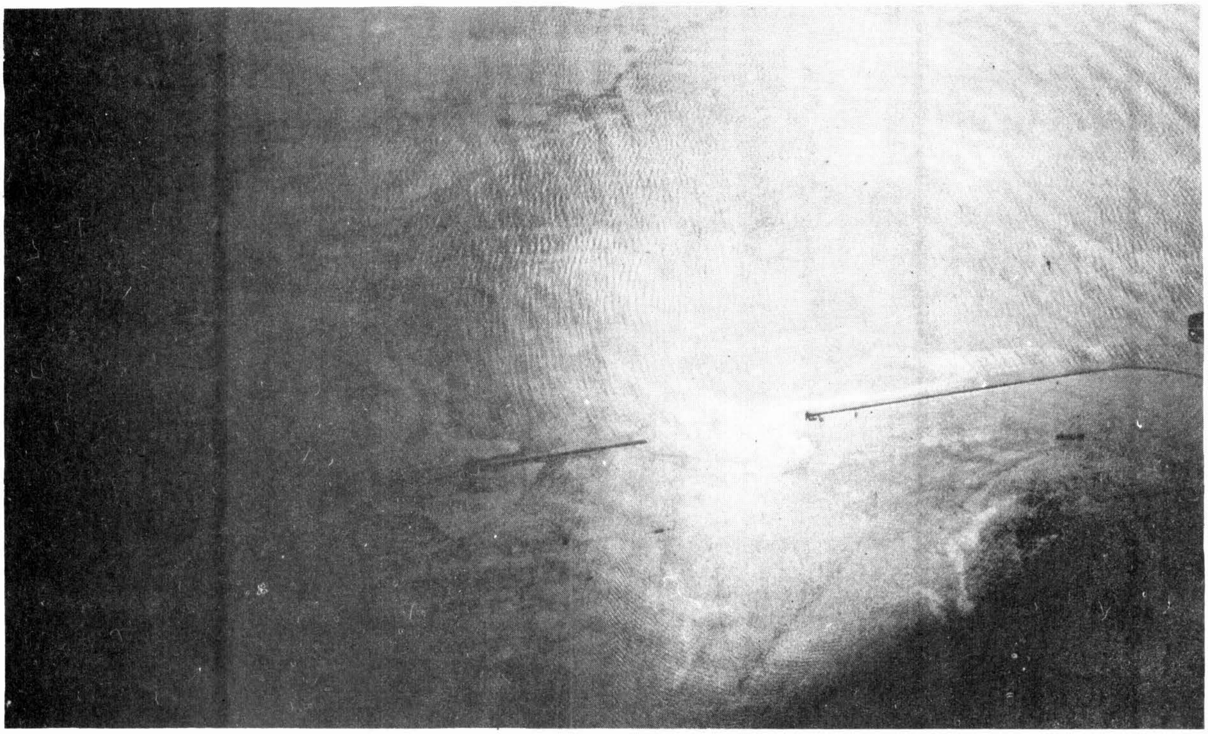


FIG. 42 WAVE PATTERN ENTERING WEST GATE

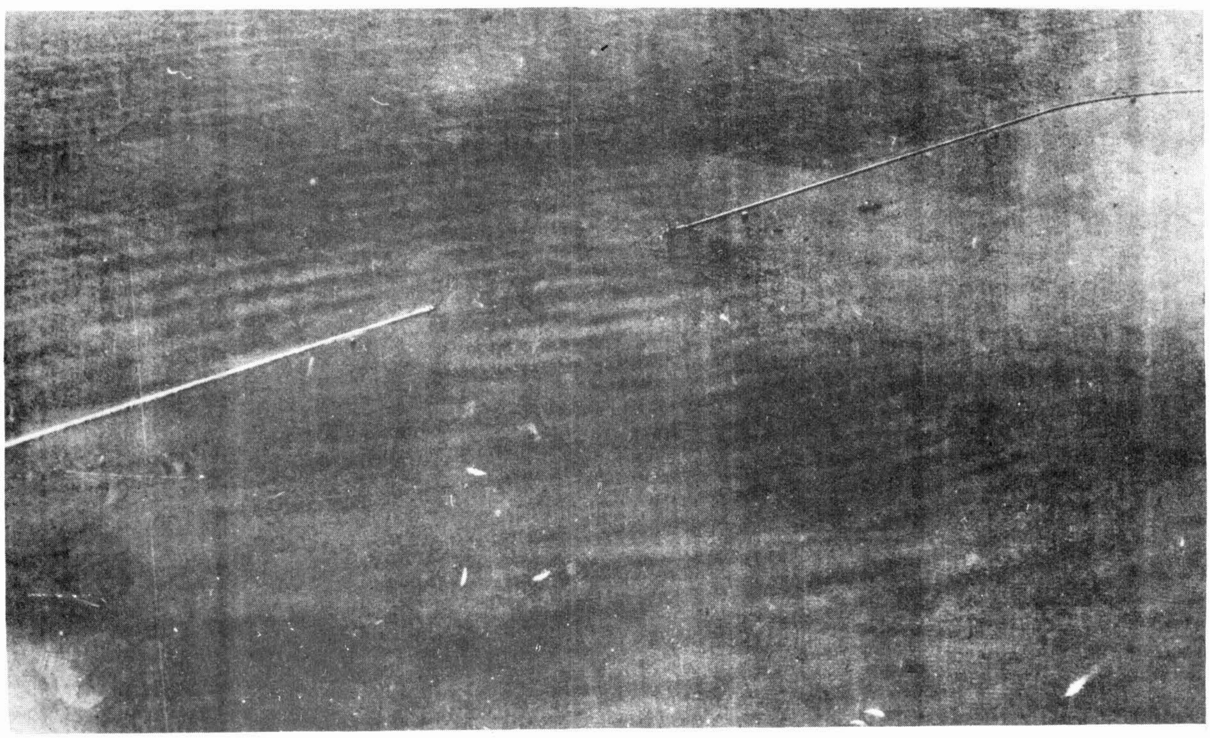


FIG. 43 WAVE PATTERN ENTERING WEST GATE

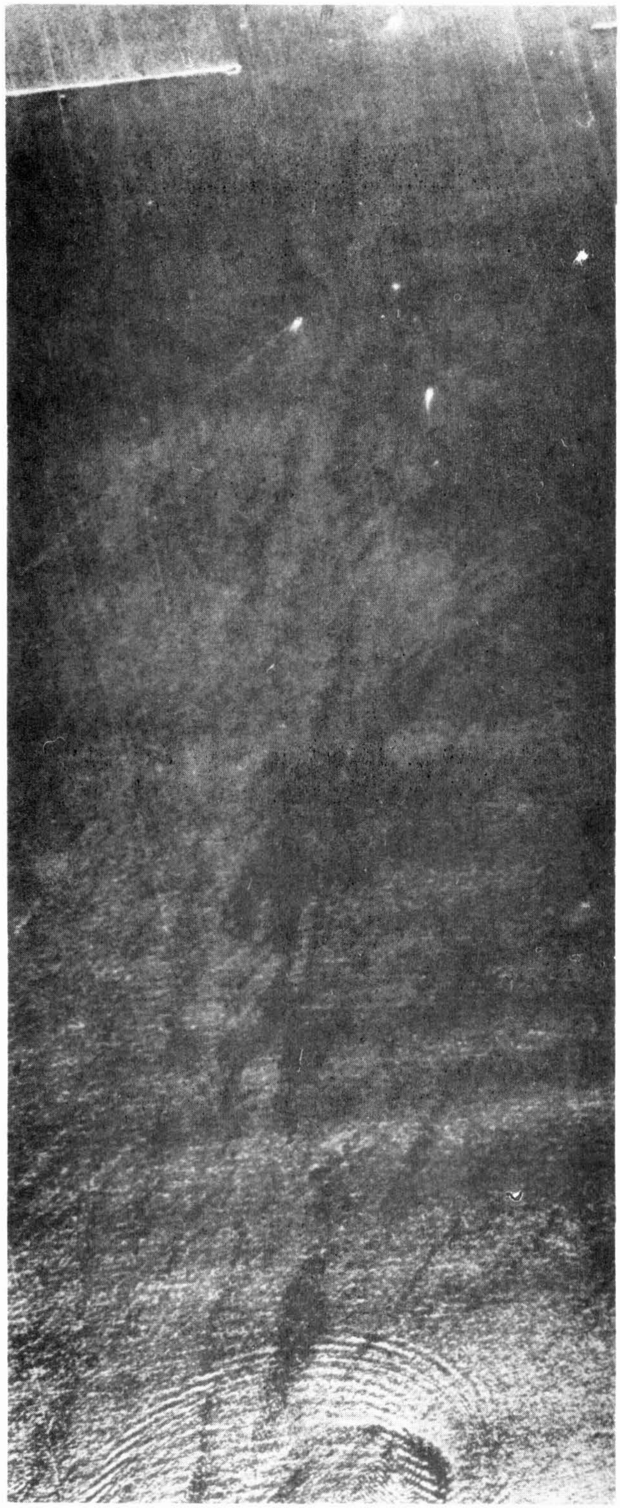
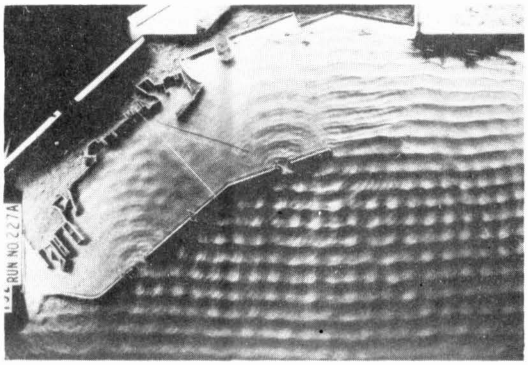
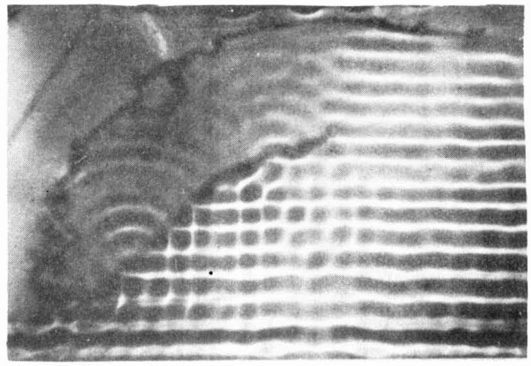


FIG. 44 WAVE PATTERN ENTERING EAST GATE

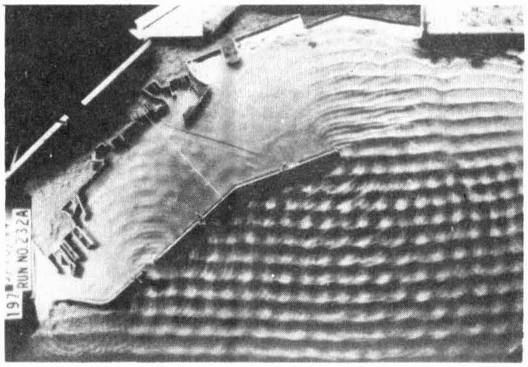
MODEL BASIN

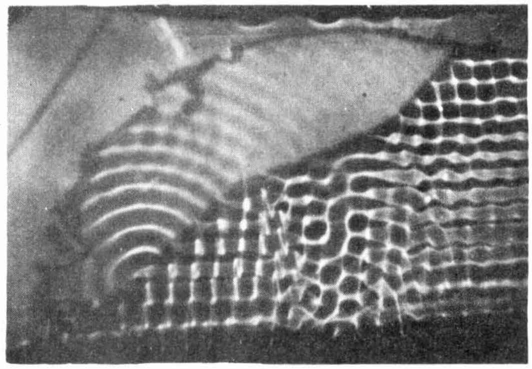
RIPPLE TANK



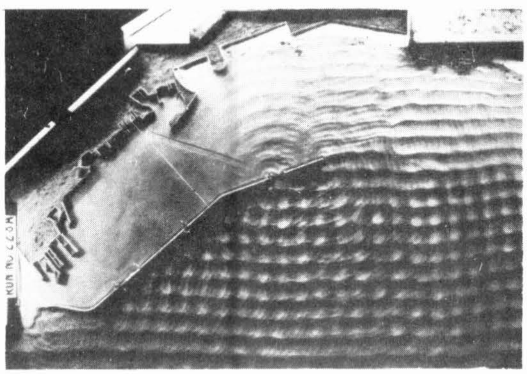


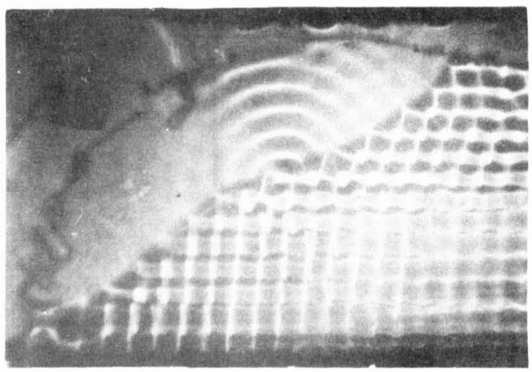
NORMAL OPENINGS



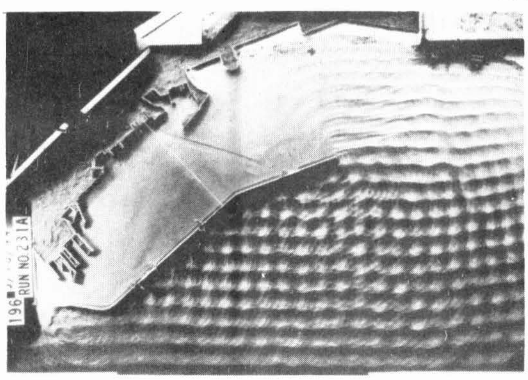


WEST GATE ALONE





EAST GATE ALONE



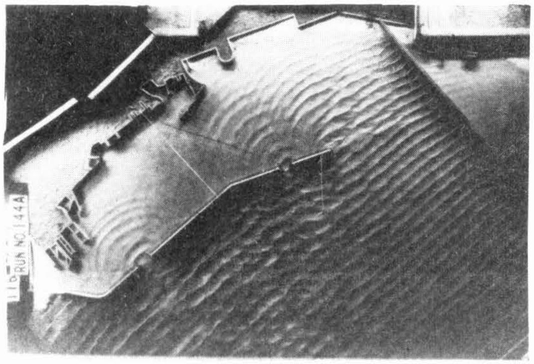
BOTH GATES CLOSED

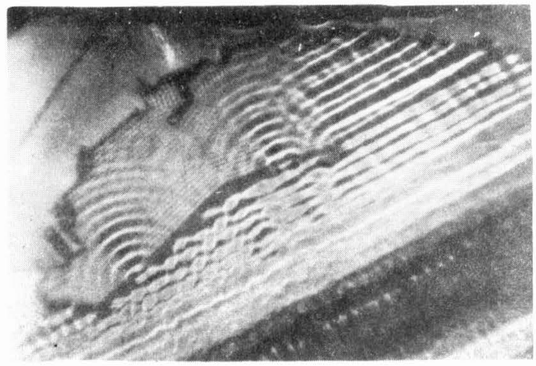
FIG. 45 WAVE PATTERNS FROM BREAKWATER OPENINGS

NORMAL WAVES FROM SOUTHWEST

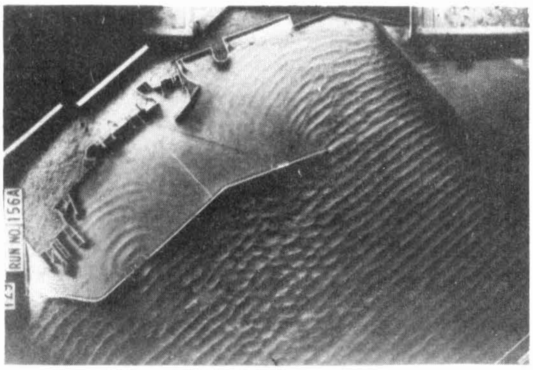
MODEL BASIN

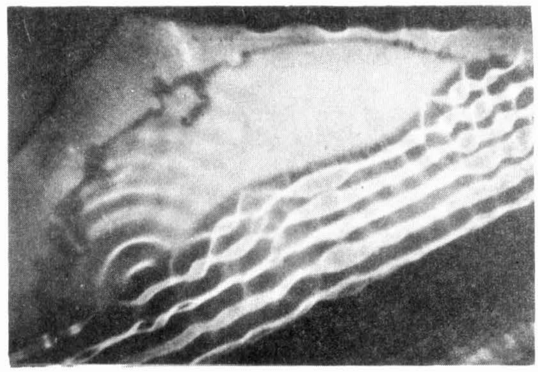
RIPPLE TANK



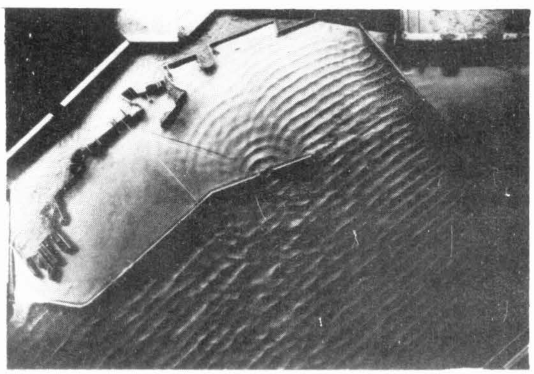


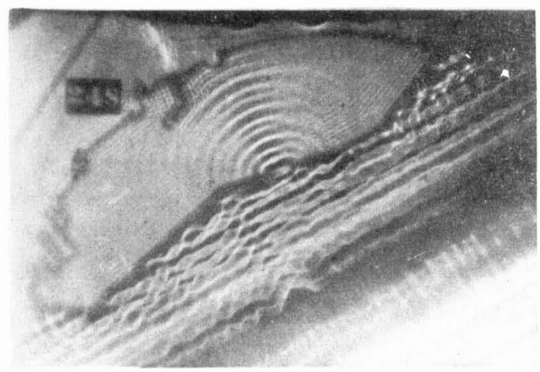
NORMAL OPENINGS



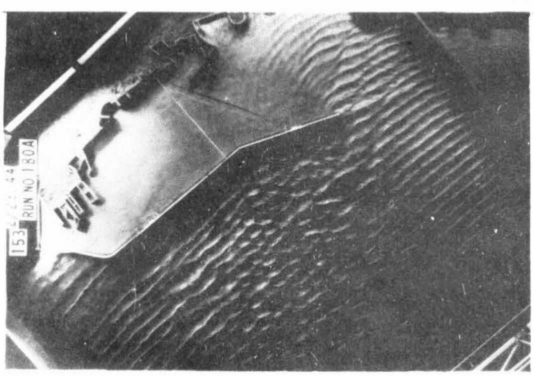


WEST GATE ALONE





EAST GATE ALONE



BOTH GATES CLOSED

FIG. 46 WAVE PATTERNS FROM BREAKWATER OPENINGS

STORM WAVES FROM SOUTHEAST

through the west gate; that a smaller percentage of the disturbance can, however, be traced to the east gate, and that practically none of the disturbance can be traced to the open east end. This is particularly clear in the picture which shows the open east end functioning alone.

<u>(b) Action of openings with storm waves.</u> Figure 46 presents a similar series of wave patterns produced by a wave train approaching the breakwater from the southeast. This is the direction from which some of the severe storm waves have been observed to come. It will be seen that with these conditions the secondary wave trains from the two gates produce nearly the same amount of disturbance, although the east gate appears to be slightly the stronger source. Again, the disturbance coming through the open east end is very small compared to the size of the opening, but with this wave direction a small amount of disturbance from this source does reach the Naval Operating Base area. It would appear that waves from the southeast are potentially the most serious ones and that the mole should be designed, if possible, to protect the enclosed basin from these conditions in which wave trains of approximately equal amplitude approach from both gates.

3. EFFECT OF MODIFICATIONS OF BREAKWATER GATES

A study was also made to amplify the pilot study from the ripple tank in which the possibility was explored of modifying the breakwater gate openings to decrease the amount of disturbance reaching the critical area. Figures 47, 48 and 49 show the resulting wave patterns. It will be seen that with the simple spurs proposed reasonably effective results were secured with the waves from the normal direction. However, the protection was much less effective from the "southeasters". This is particularly true for the east gate. In this location the only adequate protective construction extended across the 45 ft. channel and thus would present a serious inconvenience to navigation. It should be remembered that such construction offers protection only from the 600 ft. wave trains. The three minute surges and those of longer periods would probably be unaffected. Since these long period waves apparently cause the most serious disturbance at the Naval Operating Base, it was felt that these gate alterations did not offer enough promise to warrant further study at this time.

4. OPTIMUM LOCATION OF MOLE GATE

The design of the mole as proposed before this study was commenced called for the navigation opening into the protected basin to be at the eastern end of the mole near the point where the 45 ft. channel from the east gate enters the drydock area. A short study was made on this model to see whether or not any other position of the entrance would offer more protection to the inner basin. A visual inspection of other entrance locations served to demonstrate that any movement of the entrance toward the west was detrimental because it admitted more disturbance from the wave train originating at the west gate. Such

NORMAL WAVES STORM WAVES

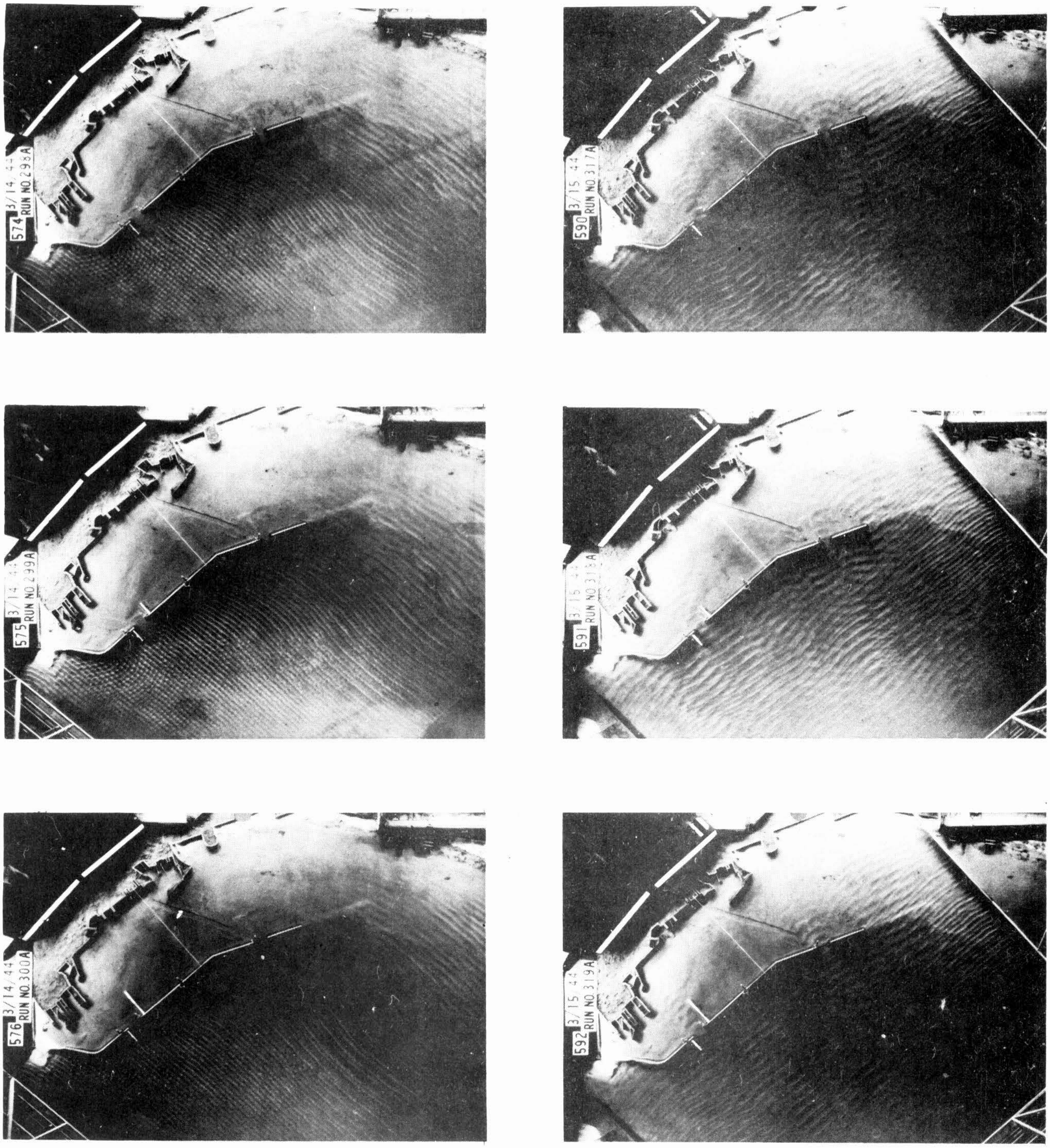


FIG. 47 POSSIBLE MODIFICATIONS OF OUTER BREAKWATER OPENINGS

NORMAL WAVES

STORM WAVES

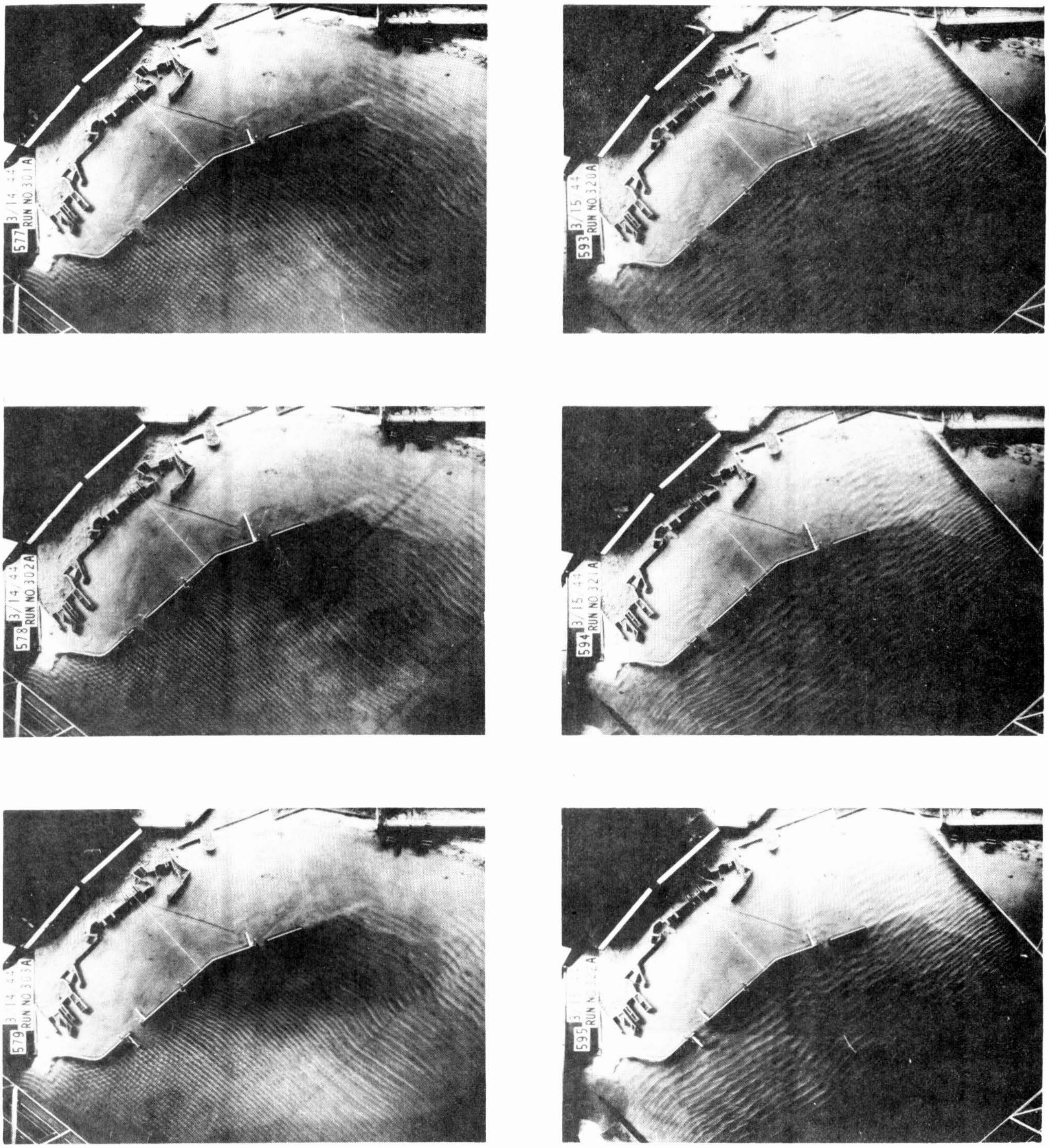


FIG. 48 POSSIBLE MODIFICATIONS OF OUTER BREAKWATER OPENINGS

a westerly movement of the entrance presumably would reduce the amount of disturbance caused by the wave trains originating at the east gate, but since these wave trains normally have lower amplitudes than those from the west gate, the total disturbance in the basin would tend to increase as the entrance was moved to the west even with wave trains coming in from both gates. It was therefore decided that the location proposed originally was the most satisfactory one and that no further gate location studies would be necessary.

NORMAL WAVES STORM WAVES

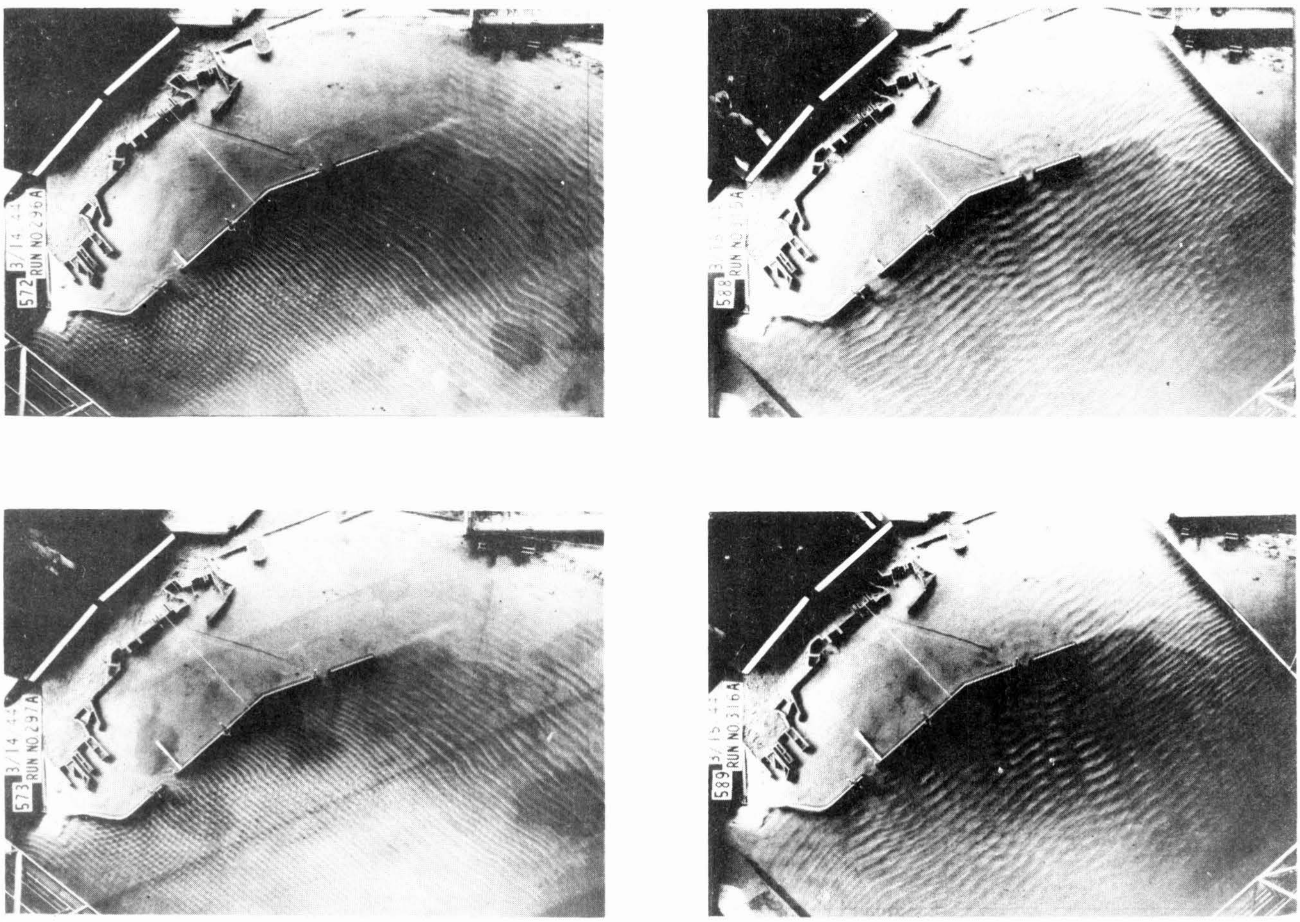


FIG. 49 POSSIBLE MODIFICATIONS OF OUTER BREAKWATER OPENINGS

VII. FINAL STUDIES IN MODEL BASIN

The basic objectives of these final studies were, as previously stated, to study the details of the mole configuration as they affect the motion of the water and to investigate the conditions to be expected inside of the mole basin after the construction has been completed.

A. OUTLINE OF EQUIPMENT

Although, in the presentation of the results of these final studies in the model basin, different models will be referred to, it should be noted that these are all modifications of a single model, since all the models cover the same area, are made to the same scale, and are studied to ascertain their reaction to the same standard series of waves and surges. They differ only as to the design of the mole and the amount and extent of the dredging in the inner basin.

1. MODEL AREA AND MODEL SCALES

Figure 50 shows a general view of the model basin with the model in place, including one of the various mole designs studied.

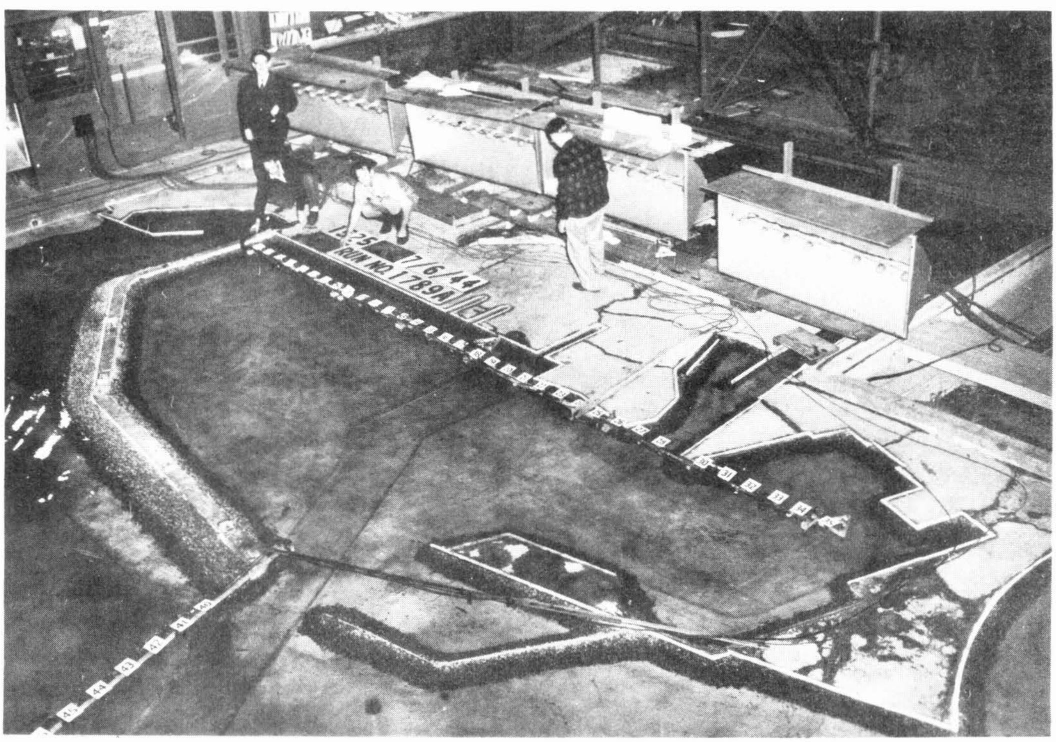


FIG. 50 GENERAL VIEW OF MODEL BASIN WITH MODEL NO. 2 IN PLACE

It will be seen that this model covers a much smaller area than Model No. 1. On the west it begins at the Naval Air Station, and on the east it ends at Los Angeles River Flood Control Channel. It was felt that this was the smallest area that could be represented in the model and still get undistorted conditions in the critical area under study. The horizontal model scale which permitted this amount of area to be represented is 1 to 480, and the corresponding vertical scale with a distortion of 1 to 2 is 1 to 240. These scales are in accordance with the discussion presented in Section IV-E (Page 44).

2. WAVE AND SURGE MACHINES

Figure 51 shows the location of the wave machine as it was installed for this series of models. It will be seen that this location of the wave machine coincides very nearly with the position of the breakwater. Since the breakwater is considered to be opaque for 600 ft. waves, the two gates are the only sources of disturbance. Hence, the long plunger of the wave machine was removed and two short plungers were substituted in its place. Their location corresponds to the position of the breakwater gates. If Figure 51 is again referred to, it will be seen that the walls of the model basin made it impossible to locate the plunger for the west gate at exactly the required position. However, it was placed directly on the line from the west gate to the critical area and, of course, normal to this line. Thus, the wave pattern from the plunger was approximately the same as it would have been if it had been located exactly at the west gate. These plungers were so constructed that they could be removed quickly and thus studies could be made of the effect of the disturbance from either gate alone or both gates simultaneously.

For the use of this model a surge machine was constructed which can produce a continuous wave train of any desired prototype amplitude and any period between one minute and twelve hours. It is also capable, within limits, of producing a motion which combines several frequencies and amplitudes.

3. MEASUREMENTS OF VERTICAL WATER MOTION

The type of information required from these studies demanded that measurements be made of the vertical amplitude of motion of the water surface, corresponding horizontal amplitudes, the determination of the wave patterns, and the amplitudes of the various components of ship motions.

The method of measurement of the vertical amplitude of the water motion was developed in conjunction with the studies on the first model. The measurement consisted essentially in the determination of the quantity of electric current flowing between two vertical wires. A constant low voltage is maintained across the two wires. The amount of current that flows is, therefore, proportional to the amount of wire surface in contact with the water and this, in turn, is directly proportional to the depth of

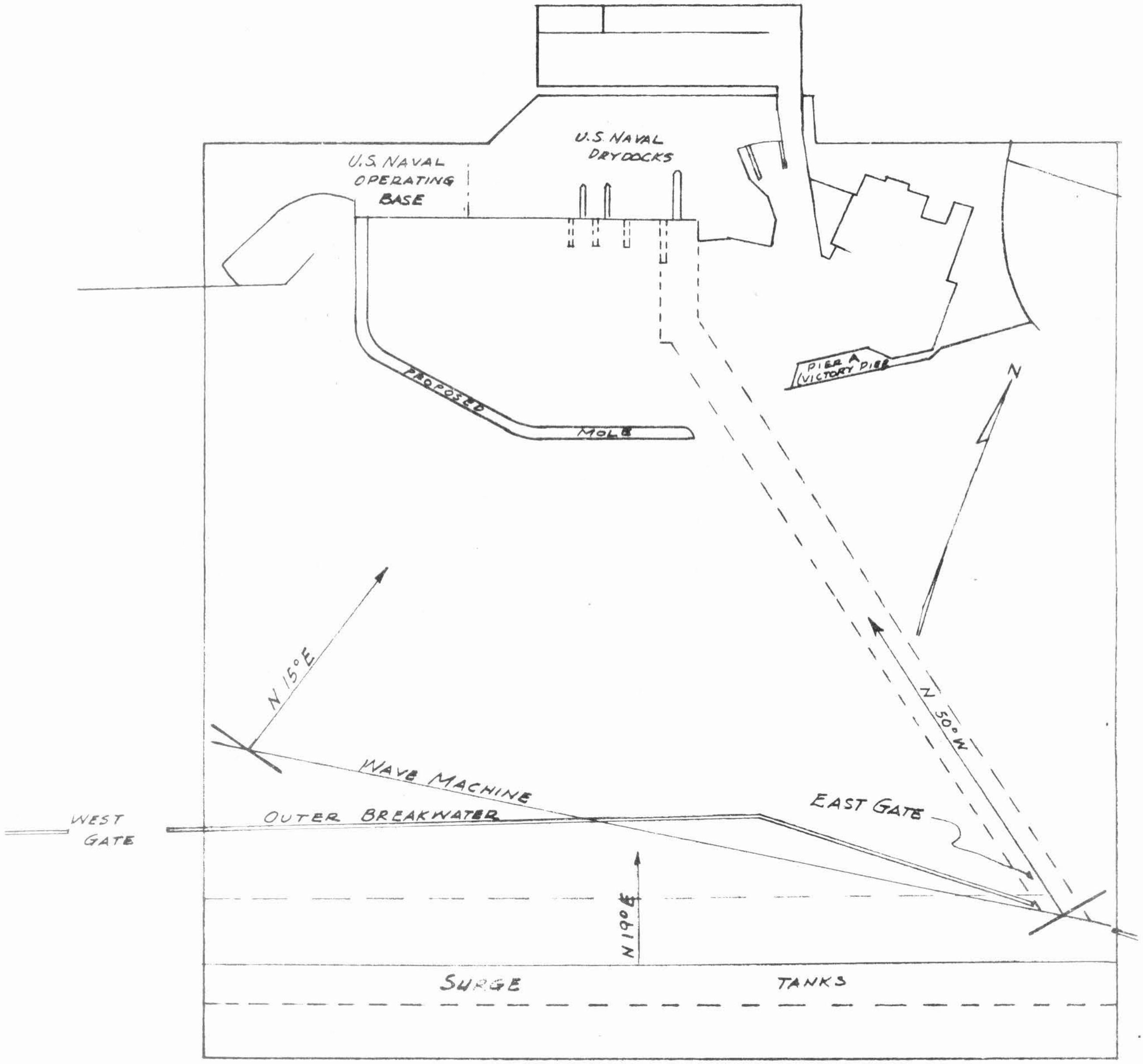


FIG. 51 DIAGRAM OF MODEL NO. 2 SHOWING POSITION OF WAVE MACHINE AND PLUNGER SECTIONS

immersion of the wires. The current passes through an oscillograph and the resulting record gives a variation of the water depth with the time. The experiments in the first model demonstrated that the standing wave pattern in the basin was so pronounced that it was impossible to get a staisfactory single measurement that would represent the performance of the basin as a whole and that instead of single measurements, it would be necessary to determine the movement of the entire surface. To accomplish this a number of the wire conductivity elements were constructed and mounted at fixed intervals on bars that could be moved from one position to another in the basin. In this way the entire area could be covered by a grid of measuring points spaced closely enough together to supply information from which contour maps of the surface motion could be constructed. Figure 52 shows two of the elements.

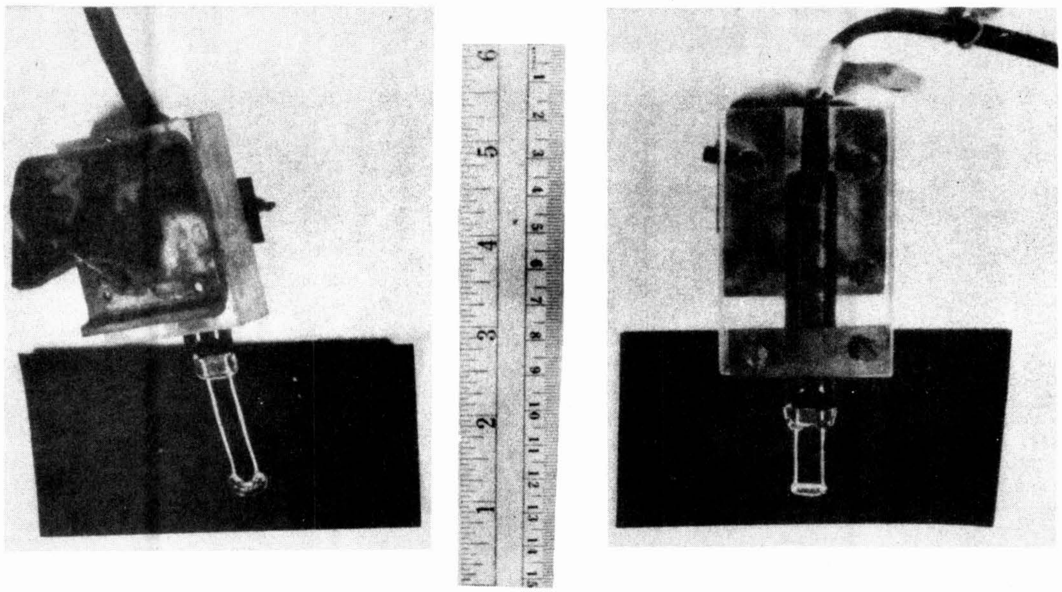


FIG. 52 CONDUCTIVITY ELEMENTS FOR MEASURING WAVE HEIGHTS

Figure 53 shows a typical record of 600 ft. wave train. Figure 54 gives a similar record for a three minute surge.

The visible wave trains were recorded photographically by the same equipment used in the first model. Figure 55 shows a photograph from this series.

4. MEASUREMENTS OF HORIZONTAL WATER MOTIONS

The horizontal motion of the water surface was also recorded photographically by introducing into the basin a large number of small floats, each of which carried a small reflecting brilliant so constructed that on a wide range of incidence angles the prism would reflect back the light towards the light source. A spotlight was mounted on the camera platform close to the lens and time exposures were taken at night, using only the illumination of this single spotlight. Figure 56 shows a photograph taken by this method. It will be seen that this indicates drift, or current, as well as the oscillation due to the individual surge.

5. MEASUREMENTS OF SHIP MOTION

It was felt that valuable information concerning the types of disturbance that caused undesirable ship movements and also

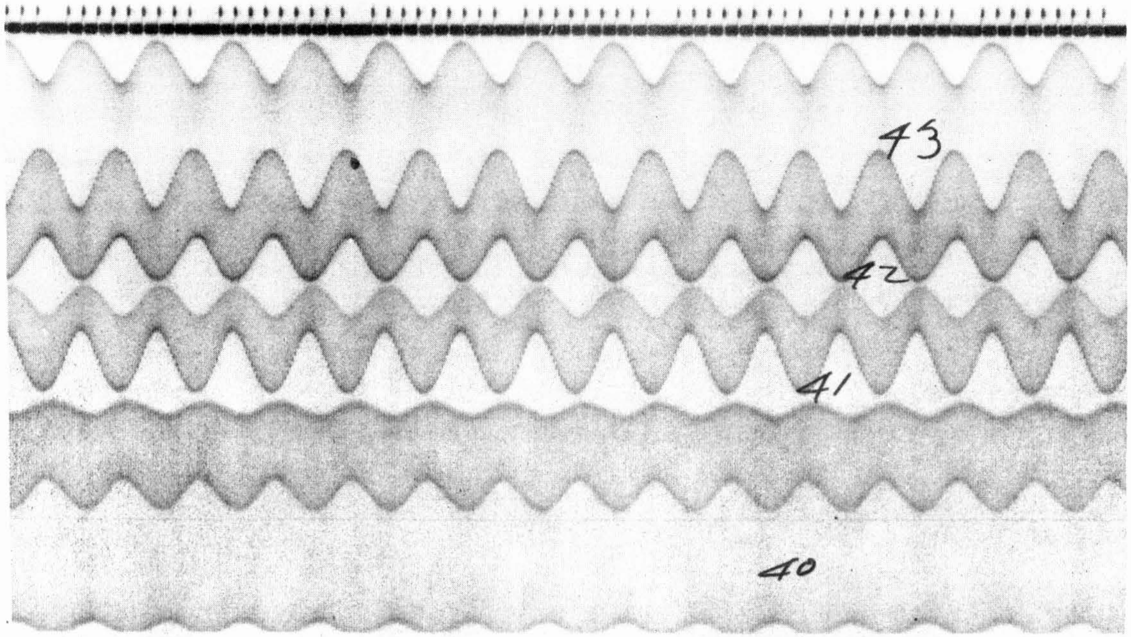


FIG. 53 TYPICAL 15 SECOND (600 FT.) WAVE RECORD

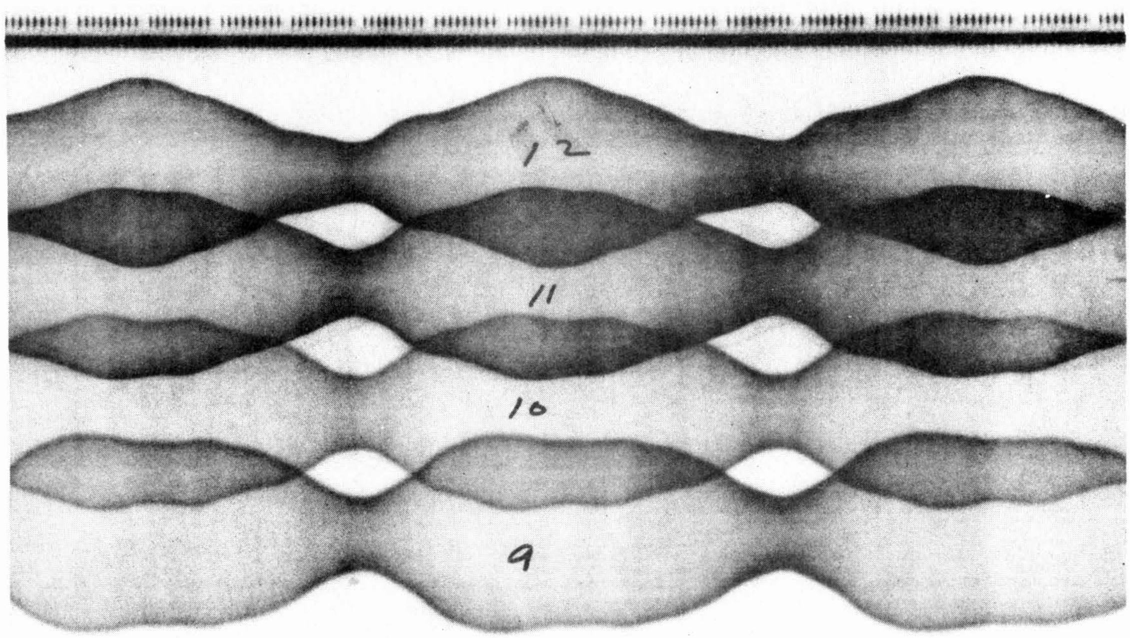


FIG. 54 TYPICAL 3 MINUTE SURGE RECORD

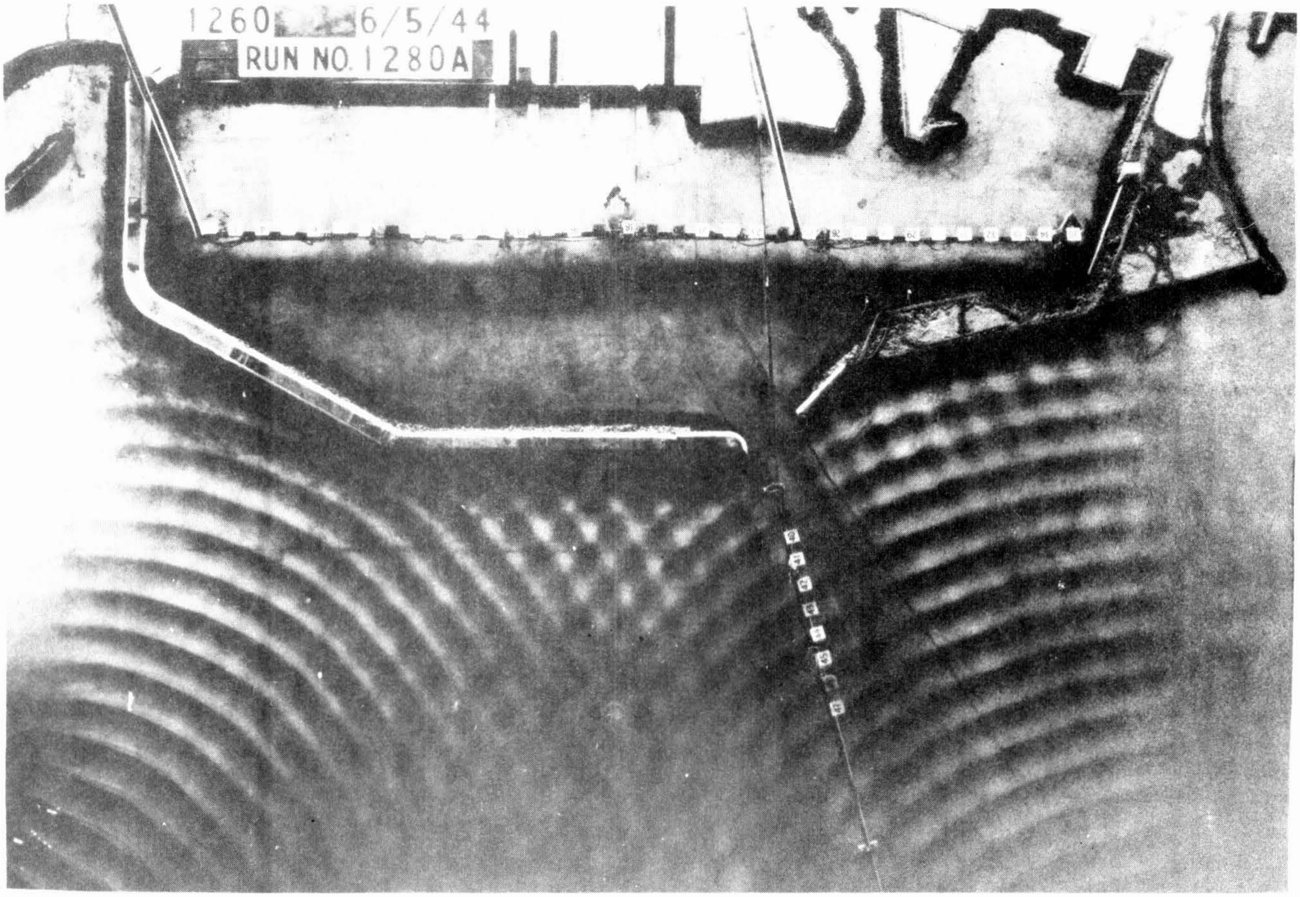


FIG. 55 MODEL NO. 2, TYPICAL WAVE PATTERN

the comparative value of the different mole configurations and internal arrangements could be obtained from a study of the motion of model ships. Consequently, several ship models were constructed and placed in the basin for observation. Comparative qualitative records were obtained by taking motion pictures of a given group of ships under varying conditions of basin operation and configuration. In an endeavor to obtain additional semi-quantitative information, one ship model thought to be the most typical of the general class that would require the facilities of the base was equipped with pointers which indicated the motion on scales fastened to the pier. A motion picture camera was used again to record the ship's movements under a variety of conditions but this time the film was used for direct measurement purposes. The installation is shown in Figure 57.

A more complete description of the model basin and its apparatus and instrument equipment will be found in Appendix II.

B. SUMMARY OF RESULTS

The following discussion of the results will be found divided into two parts under the headings of Model 2 and Model 3,

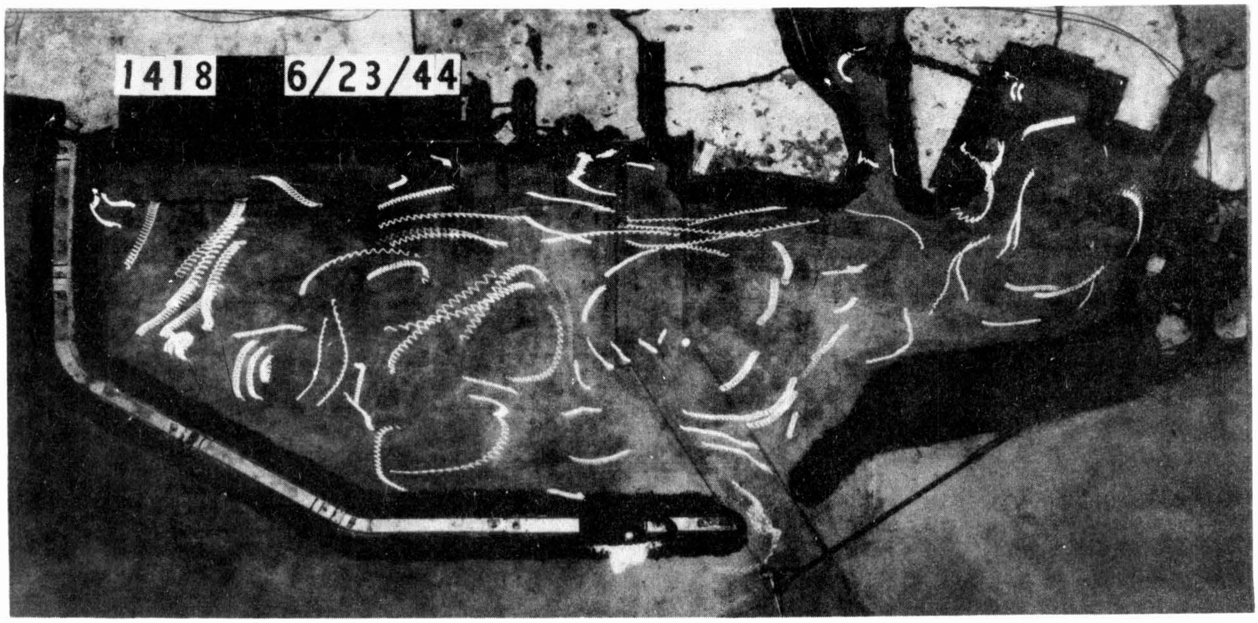


FIG. 56 MODEL NO. 2, TYPICAL 3 MINUTE SURGE REFLECTOR PATTERN

respectively. The basic difference between the two models, with the exception of the various mole configurations and additional structures, is that in Model 2 the basin to be enclosed by the mole was left at the original depth, which was approximately 35 ft. except for a small area with a depth of 45 ft. in the vicinity of Pier 1 and the large drydocks; whereas, in Model 3 nearly the entire area of the basin is dredged to 45 ft. depth. The extent of this basic difference is shown in Figure 58.

1. RESULTS OF STUDIES OF MODEL 2

The following summaries of the results from studies of this model are grouped together according to subject. Therefore, the order of presentation does not necessarily agree with the order in which the studies were made.

(a) Standardization of wave trains. In the description of the wave machine used in this set of tests it was stated that two plungers were used, one to produce the wave train originating at the west gate and one for that coming from the east gate, and that these plungers were constructed so that they could be removed readily for study of the effects of the individual trains. A short investigation was carried out to determine the relative magnitude of the disturbances coming from each of the two gates. This was done on the model with the mole in place and with the eight measuring elements grouped in two different positions, first in the 45 ft. channel as shown in Figure 59 and second, in the drydock area as shown in Figure 60. This was carried out for three

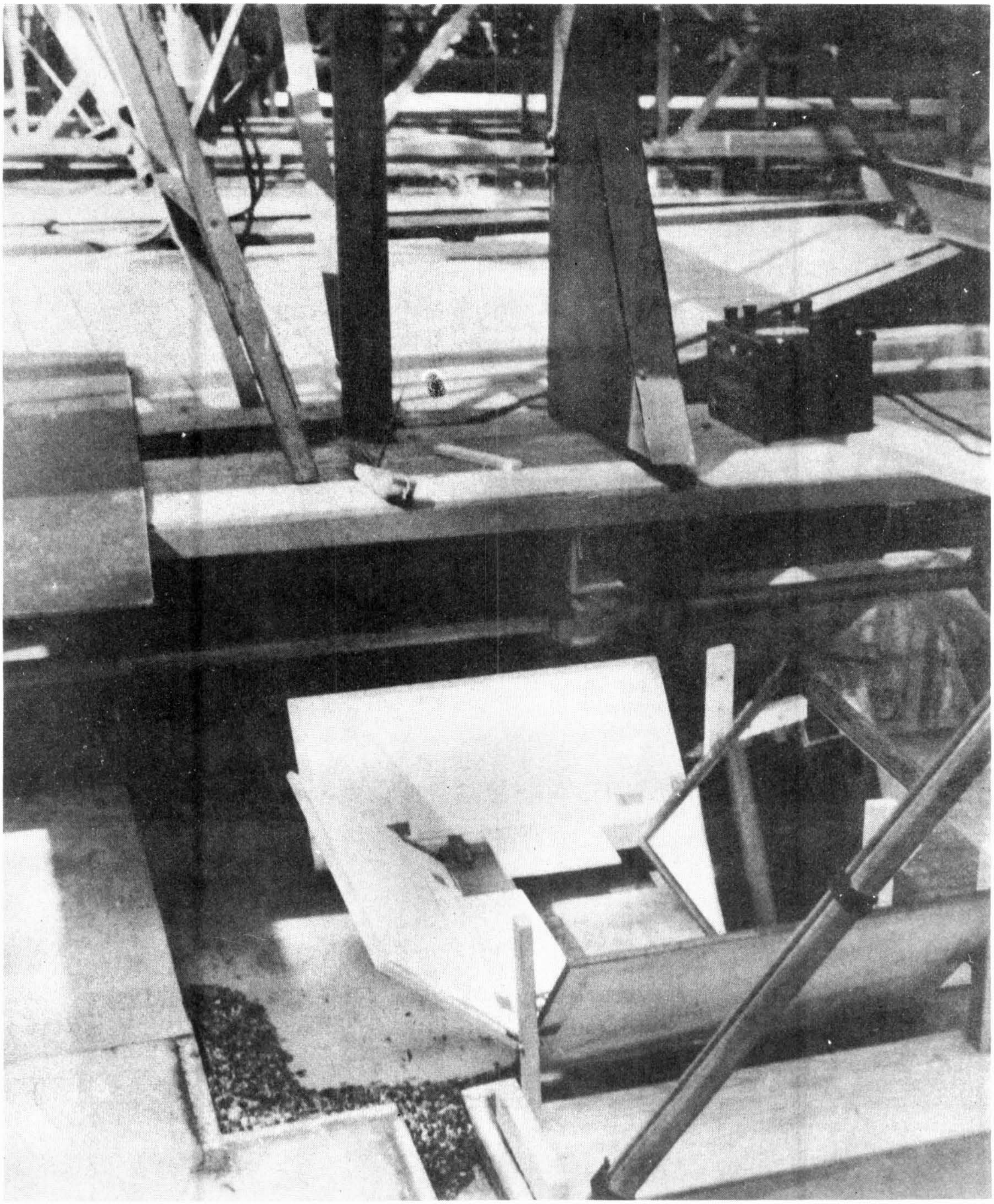
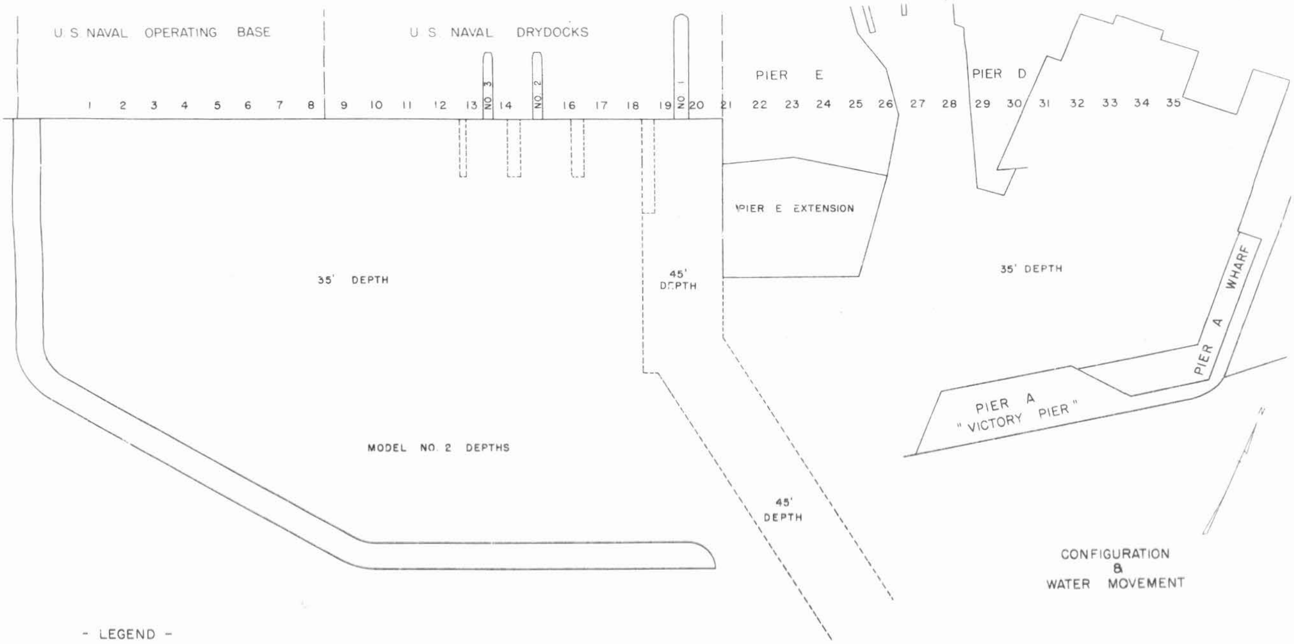
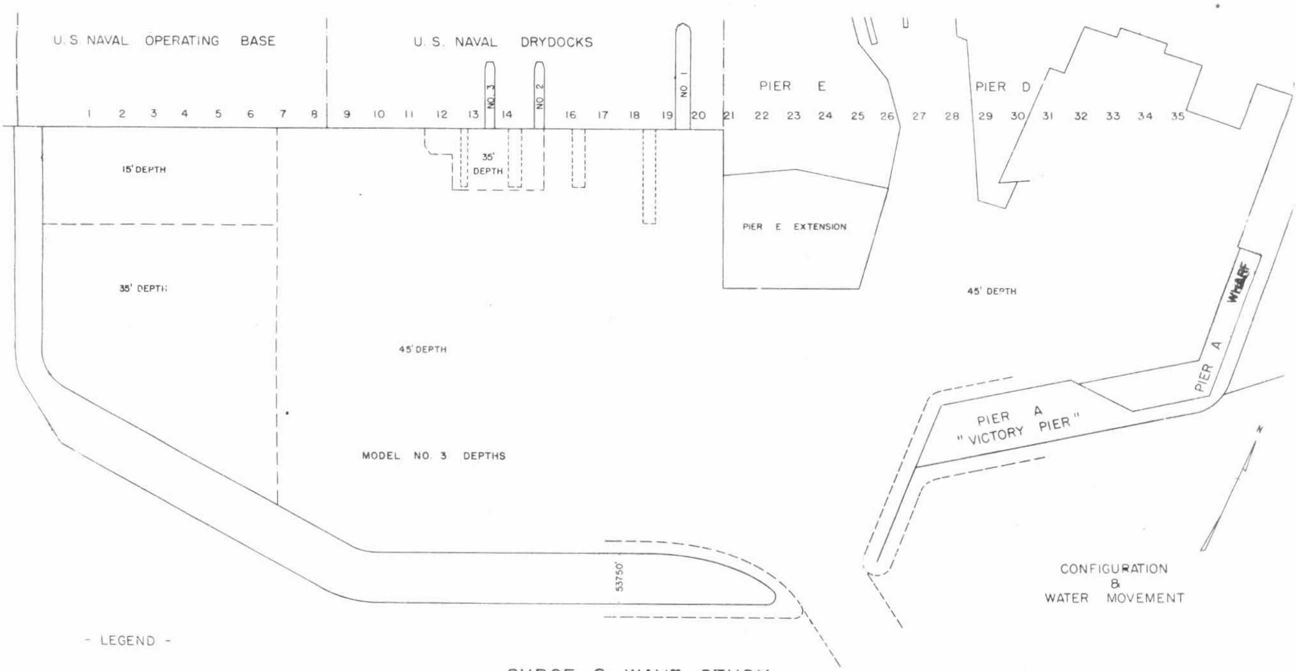


FIG. 57 GENERAL VIEW OF EQUIPMENT USED FOR MEASURING SHIP MOVEMENTS



SURGE & WAVE STUDY
U. S. NAVAL OPERATING BASE & VICINITY
LOS ANGELES - LONG BEACH HARBOR AREA
MODEL NO. 2



SURGE & WAVE STUDY
U. S. NAVAL OPERATING BASE & VICINITY
LOS ANGELES - LONG BEACH HARBOR AREA
MODEL NO. 3

FIG. 58 SKETCHES SHOWING MOLE BASIN DEPTHS

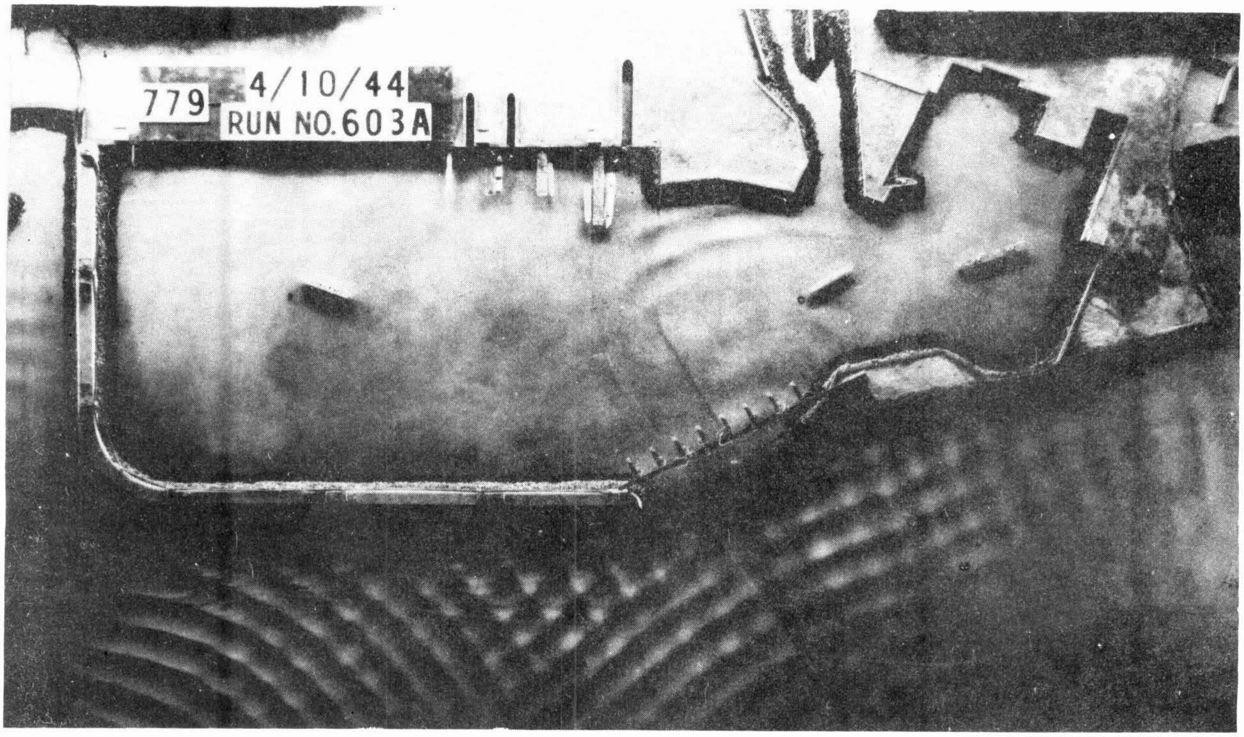


FIG. 59 LOCATION OF 8 GRID ELEMENTS ACROSS 45 FT. CHANNEL

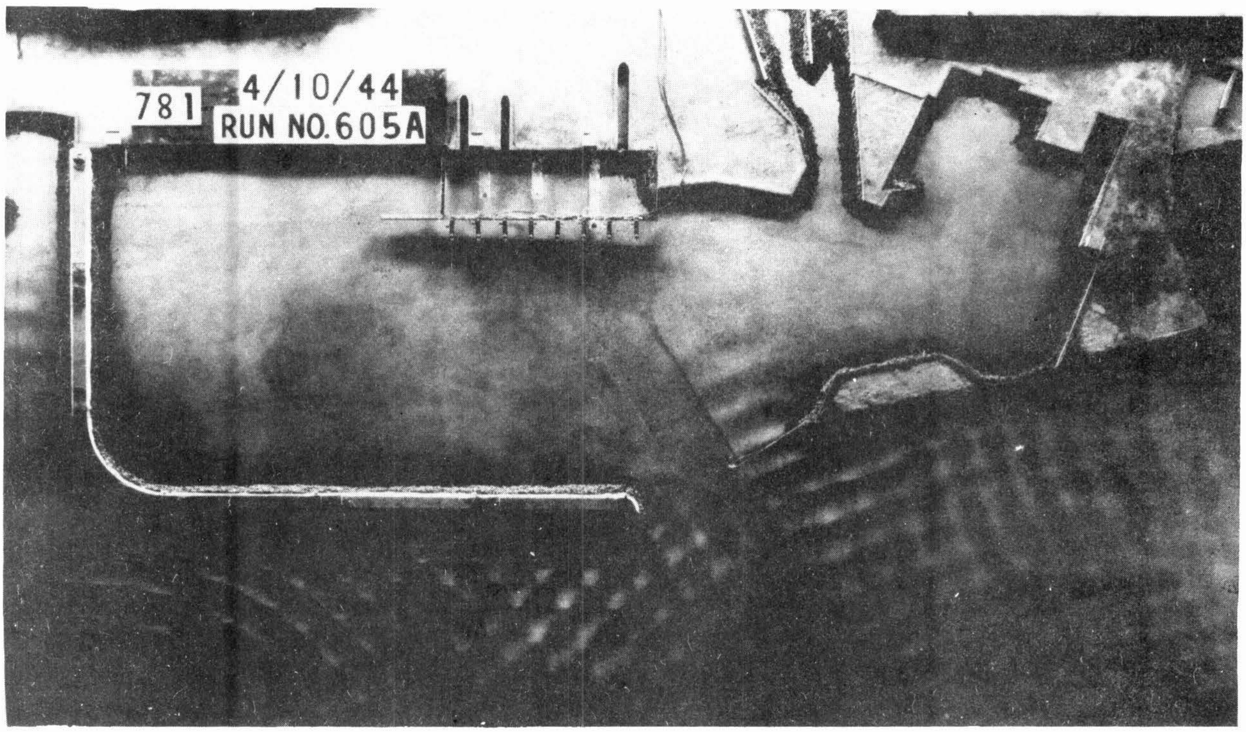


FIG. 60 LOCATION OF 8 GRID ELEMENTS IN DRYDOCK AREA

wave frequencies and measurements were made with waves from each gate alone and from the two gates simultaneously. Figure 61 shows the results of this study for the elements in the first position, and Figure 62 is a similar analysis of the results from the elements in the drydock area. It will be seen that the relative effect varied considerably with the frequency but that the disturbance produced by both gates operating together was the most severe. In fact, in most cases it was roughly the same as the sum of the disturbance from the two individual gates. It was, therefore, decided to standardize on the combined wave train from both gates as the normal test condition.

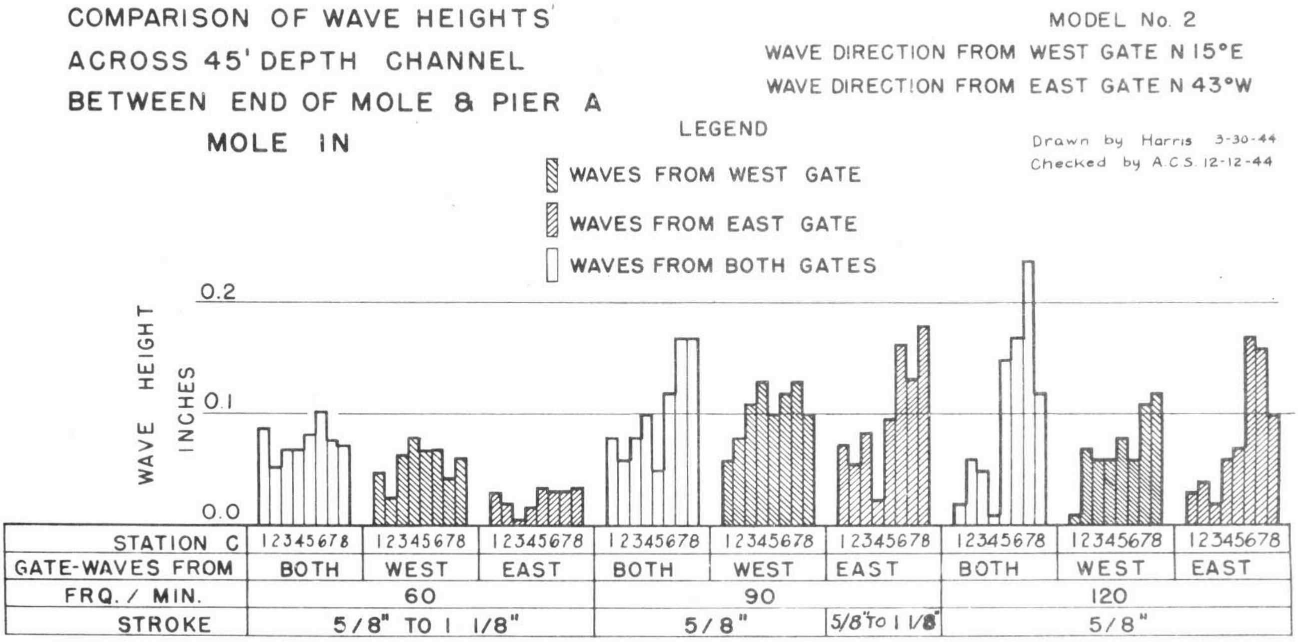


FIG. 61

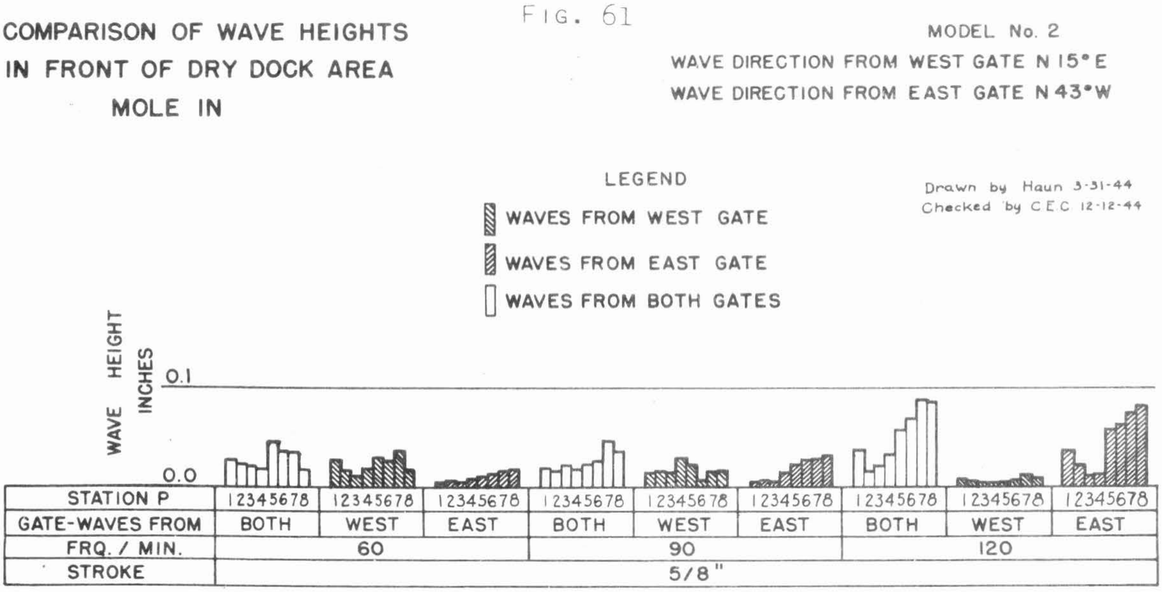


FIG. 62

<u>(b) Frequency response of the basin.</u> In Section IV-D it was pointed out that any basin will have a fundamental frequencies and a series of harmonics. Therefore, in the case of the construction of a new basin such as that enclosed by the mole, it is very important to explore the frequency response characteristics of the basin to ascertain the fewquencies at which it will oscillate in resonance with the exciting wave train and the relative amplitude of the oscillations in the basin as compared to those of the exciting wave. In order to determine these characteristics for the Naval Operating Base basin, a series of runs was made in which the magnitude of the disturbance within the basin was compared to that of the exciting wave train. The period of this exciting wave train was varied from a minimum of ten seconds to a maximum of fifteen minutes. These studies were carried out for both the 600 ft. and 2000 ft. gate openings, measured from toe to toe of slope. Parallel lines of measuring elements were installed in the basin and a similar string of elements was placed outside of the basin, parallel with the direction of travel of the exciting wave train. The results are shown in Figure 63. The ordinate represents the ratio of the maximum amplitude of the motion within the basin to that of the exciting wave. It will be observed that there are definite resonance points. The largest response was obtained for the six minute surge. A visual examination of the basin during these tests showed that with a six minute exciting wave train the basin was oscillating longitudinally with two waves traveling in opposite directions. This corresponds to the first harmonic of the basin and indicates that the fundamental is a twelve minute period. However, the results of the twelve minute test show that the basin had a lower response to this period. Visual observation again indicated the reason, which is really obvious from a consideration of the basin configuration. The gate opens into the basin at a point on one side not far from the center of the longitudinal dimension. When a wave crest enters through the opening it tends to spread in both directions, i.e., wave crests start toward both ends of the basin. In other words, a single entering wave produces two disturbances traveling in opposite directions in the basin. If the wave enters at twelve minute intervals, it still does the same thing. This means that the twelve minute wave within the basin makes one round trip before it receives a new impulse from the gate since the time of travel for one round trip is twelve minutes. (Round trip = 24,000 $\pm$ ft., wave velocity for 35 ft. depth = 34.6 ft. per second -- Time = 720 $\pm$ seconds). Thus, with the twelve minute exciting wave the disturbance within the basin has the damping of one complete round trip between each addition of energy, and thus the basin response is diminished. With a six minute wave each traveling wave within the basin gets a replenishment of its energy twice for each round trip and thus the basin gives the maximum response.

The three minute period is seen to be a resonant point as well. However, visual observation shows that the pattern movement within the basin has been altered radically because with this shorter period an oscillation across the width of the basin is possible as well as one in the longitudinal direction. Therefore, the pattern is more complicated and the energy is more subdivided,

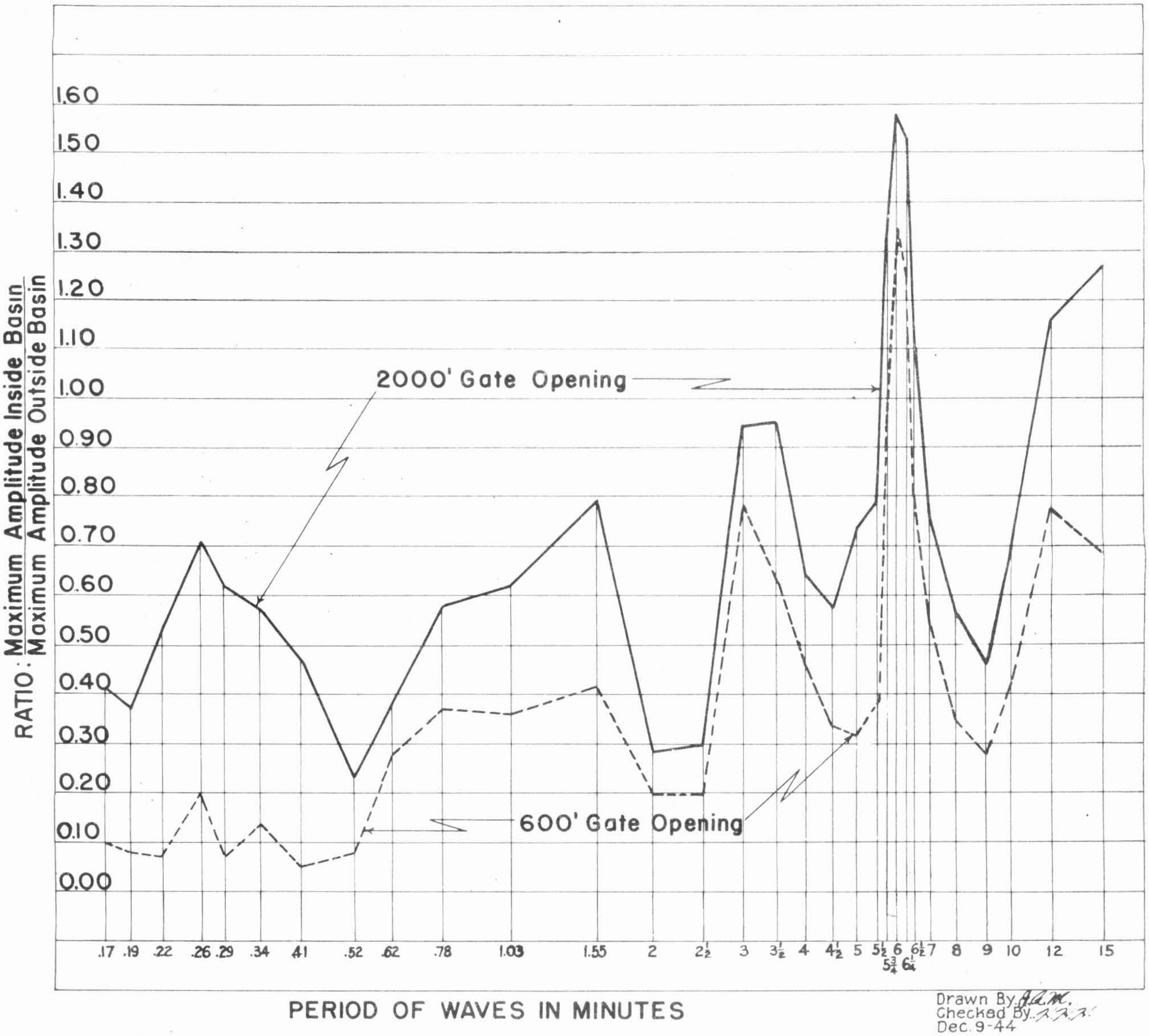


FIG. 63 FREQUENCY RESPONSE CURVES OF BASIN - 600 FT. AND 2,000 FT. GATE OPENINGS

with the result that the maximum amplitudes are lower than they are for the six minute frequency. Figures 64 and 65 show the typical contour maps of the surface motion of the basin for both the six minute and three minute frequencies. The numbers on the contour lines indicate the height of the waves to some convenient length scale. The heights of the standard waves used in the model to represent the fifteen second, three minute and six minute waves are approximately the same. Therefore, when the heights of the waves of the various frequencies inside the mole are the same, the amount of reduction or attenuation in wave height is also the same, although the amount of the attenuation is not given by the figures on the contour maps.

It will be observed that as the periods grow shorter, the sharpness of the resonance becomes less. The result is that in the period that contains normal 600 ft. waves, i.e., the period range from twelve to twenty seconds, the basin response is about constant. This is to be expected both because the harmonic frequencies are getting closer and closer together and because, due to the irregular shape of the basin, there are more and more possible paths for oscillations as the wave length gets shorter and shorter. This means that the incoming energy from the exciting wave train is split up so as to maintain more and more individual oscillations. Thus, the standing wave pattern becomes increasingly complicated but the maximum amplitude remains smaller than that for the simpler modes of oscillation. Figure 66 is a typical contour map of the surface motion for the fifteen second waves.

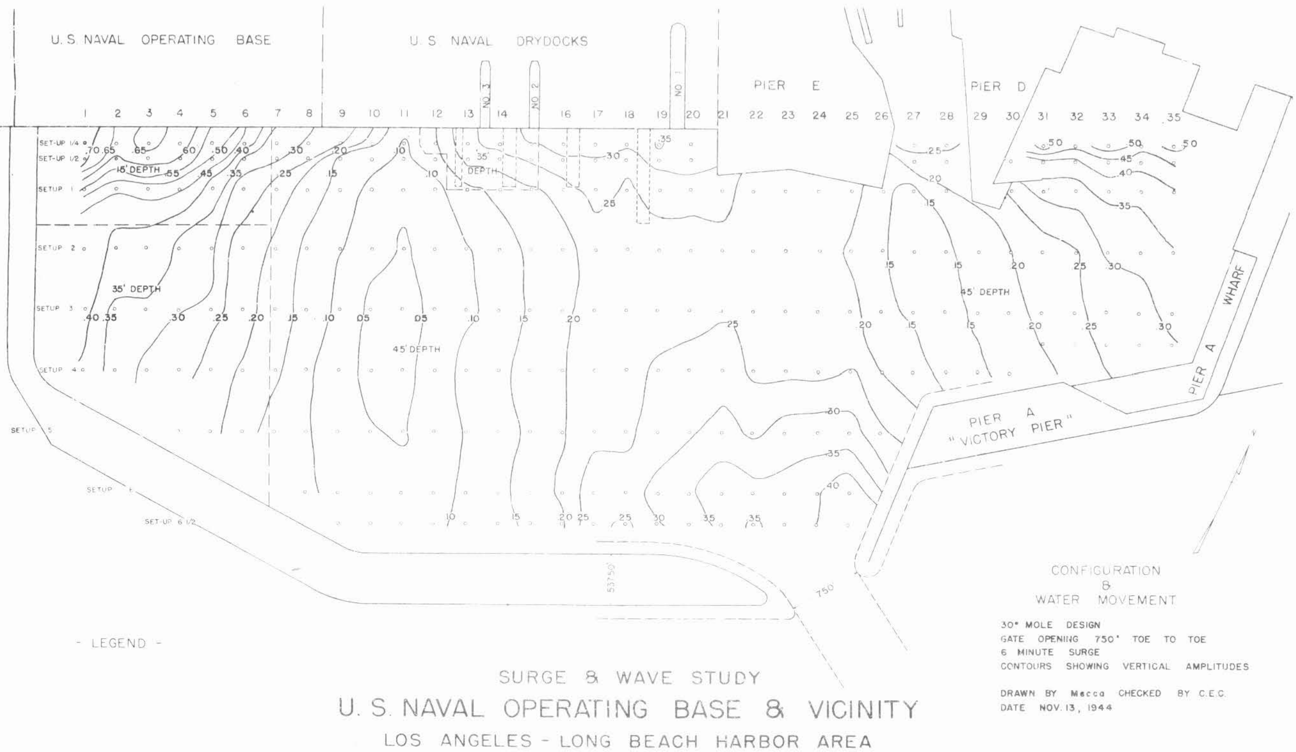


FIG. 64 VERTICAL WATER MOVEMENTS CAUSED BY 6 MINUTE SURGE

In evaluating the significance of the results of this frequency response study, it must be remembered that they are useful only for comparison purposes since they represent the behaviour of one specific configuration of mole and basin as it responds to the range of frequency. This configuration does not necessarily correspond with that of the final construction. Furthermore, changes in the structures within the basin, i.e., drydocks, opaque piers, fills, etc., may modify response to the shorter periods but they will have less effect on the long ones because their dimensions are necessarily small as compared with that

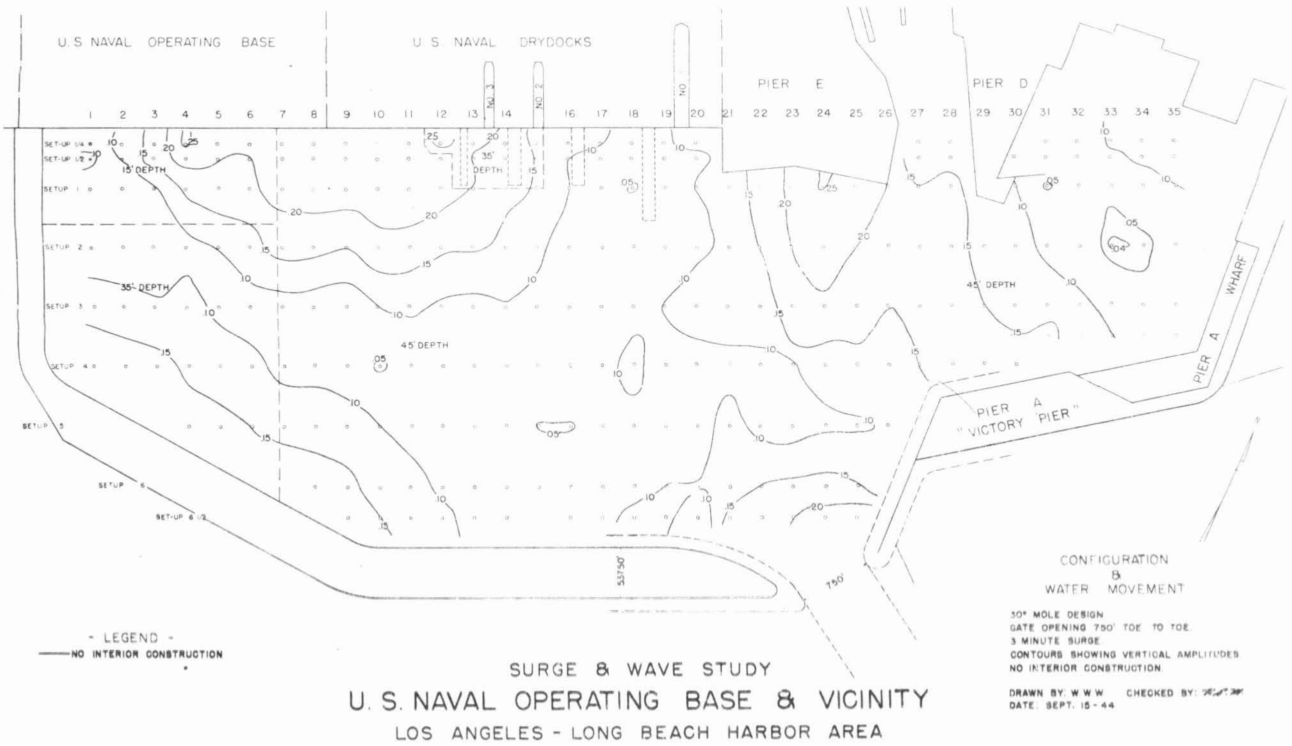


FIG. 65 VERTICAL WATER MOVEMENTS CAUSED BY 3 MINUTE SURGE

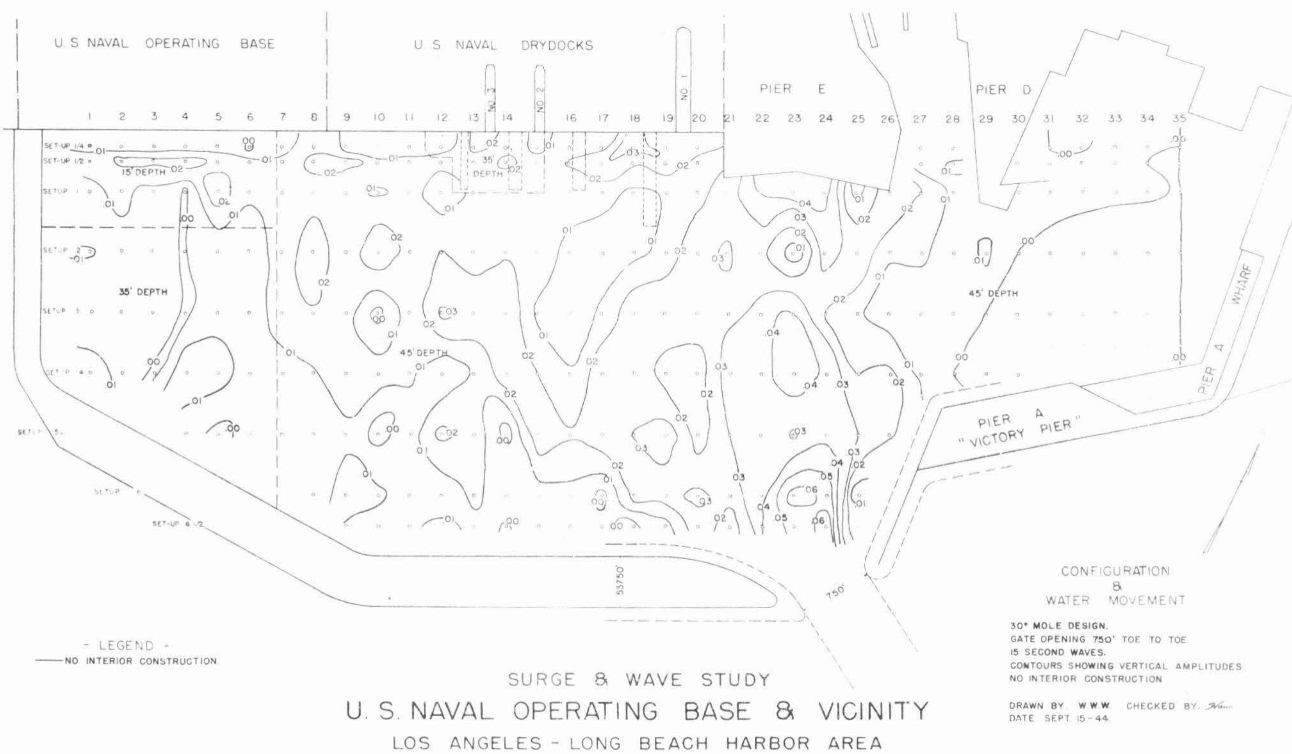


FIG. 66 VERTICAL WATER MOVEMENTS CAUSED BY 15 SECOND WAVES

of the basin itself. The significant conclusion from this study is that the most dangerous period which external disturbances can have to excite motion in the basin is about six minutes and that no other period, whether longer or shorter, will create such a large disturbance. Disturbances having periods between two and four minutes will also produce comparatively large movements in the basin, but all periods shorter than this will produce correspondingly lower disturbances. It is interesting to note that there are distinct maxima for one and one-half minute and for fifteen second periods. Field records seem to indicate the possibility of the existence of disturbances with six minute periods but their occurrence seems to be quite rare. Three minute disturbances, however, are not uncommon, and it is probable that they will become the most important remaining source of undesirable water motion in the basin after the mole is completed. The damping of the basin is fairly high for all frequencies as is shown by the fact that the maximum amplification factor observed for the widest gate and the frequency corresponding to the fundamental mode of motion was only 1.6. The fact that the fundamental period of the basin does lie within the range of frequencies of the disturbances that are known to exist in the outer harbor means that there will always be the possibility of large disturbances within the basin which probably will seem especially severe in comparison with the improved conditions for all normal wave and surge periods that will exist within the basin after the mole is completed.

(c) Mole configuration. At the time this study was started, a preliminary design had been prepared of a proposed mole configuration. The general outline of this design can be seen in Figure 67. By the time that this section of the study was being carried out this design had been modified to the shape seen in Figure 68. It will be observed that the main difference between the two designs consists in the moving of the parallel section 382 ft. seaward, which increases the width of the enclosed basin from 4070 ft. to 4452 ft., and in the elimination of the arm of the mole inclined at 15 degrees to the shoreline. One of the basic objectives of Model 2 was to compare these two designs and to explore other configurations which would result in a working basin area of approximately the same size as the second design. The purpose of this exploration was to see if a more favorable shape could be found for the mole configuration, i.e., one that would result in a quieter basin.

(1) Preliminary investigation of mole configurations. In all, fifteen different mole configurations were tested. A comparison of their outlines will be seen in Figure 69. In each case, the outline of the Standard Mole is shown. All of these configurations except the Standard Design were tested with the same gate opening (600 ft. toe to toe of slope) and with wave trains originating at both the east and west gates. In the following discussion of the results, the term Original Design will refer to the first design proposed by the Navy with the parallel leg 4070 ft. from shore, and the Standard Design will refer to the second design proposed with the parallel leg 4452 ft. from shore. Comparative tests were run not only with the waves, but with two

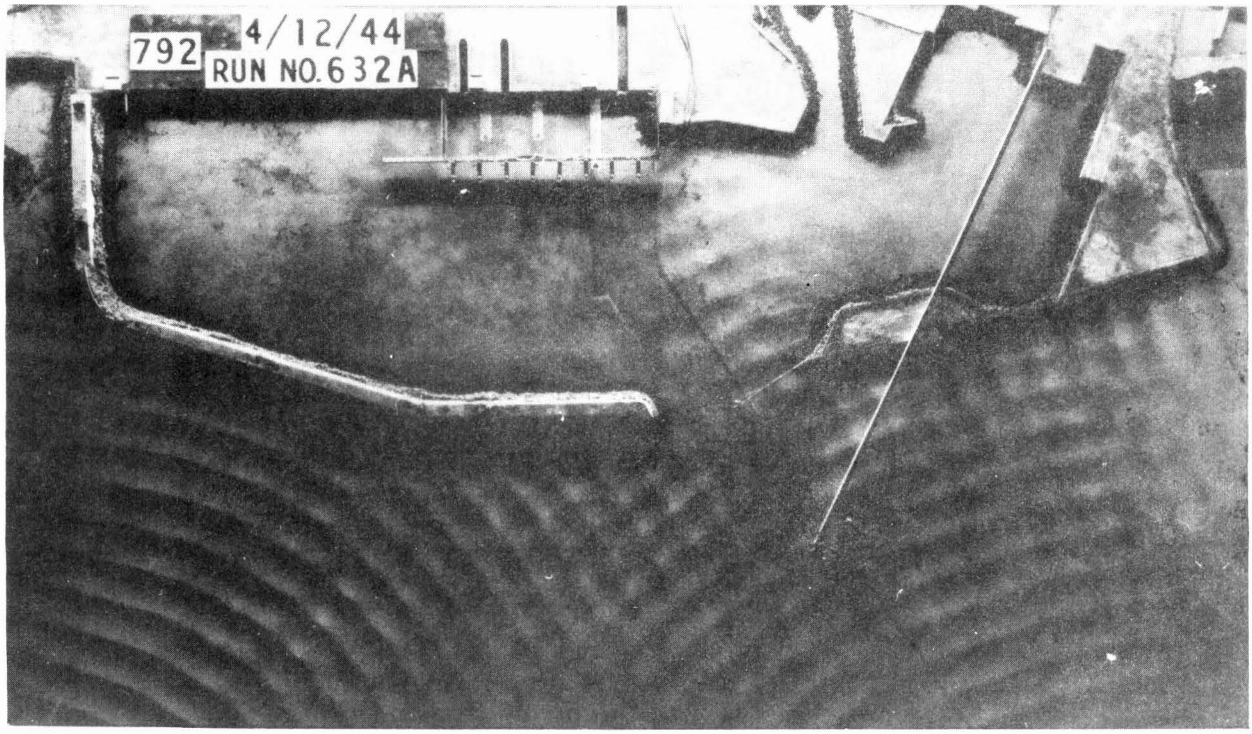


FIG. 67 ORIGINAL DESIGN OF MOLE

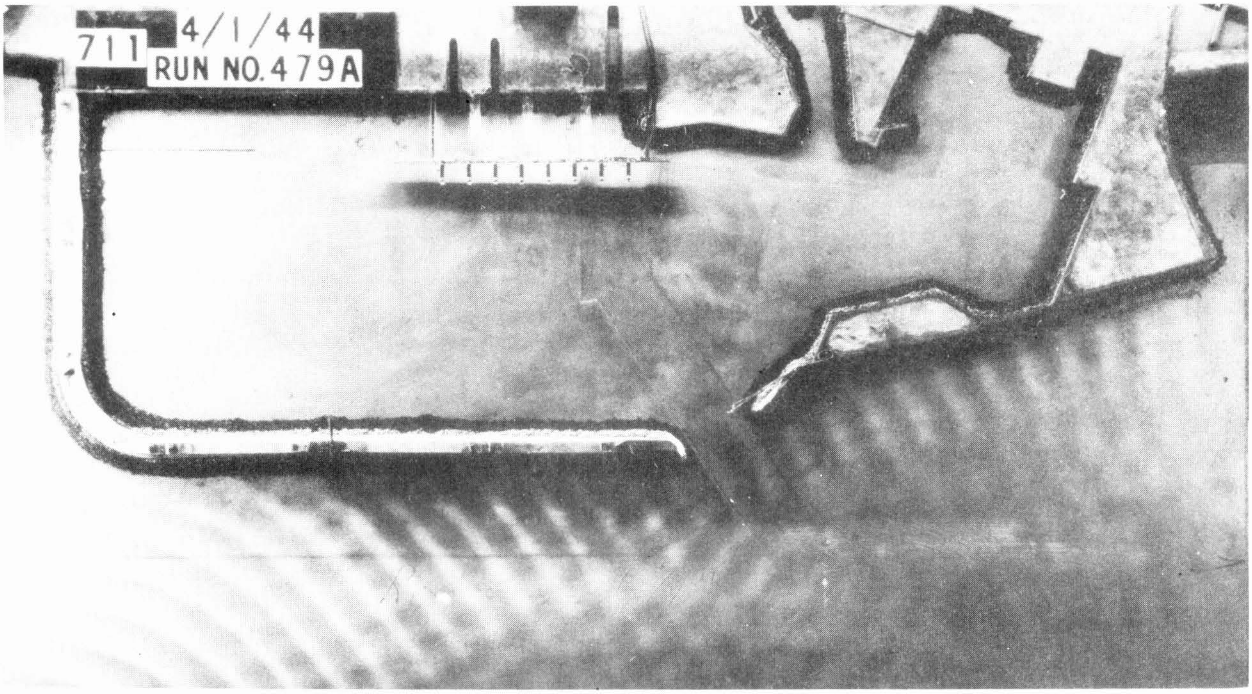


FIG. 68 STANDARD DESIGN OF MOLE

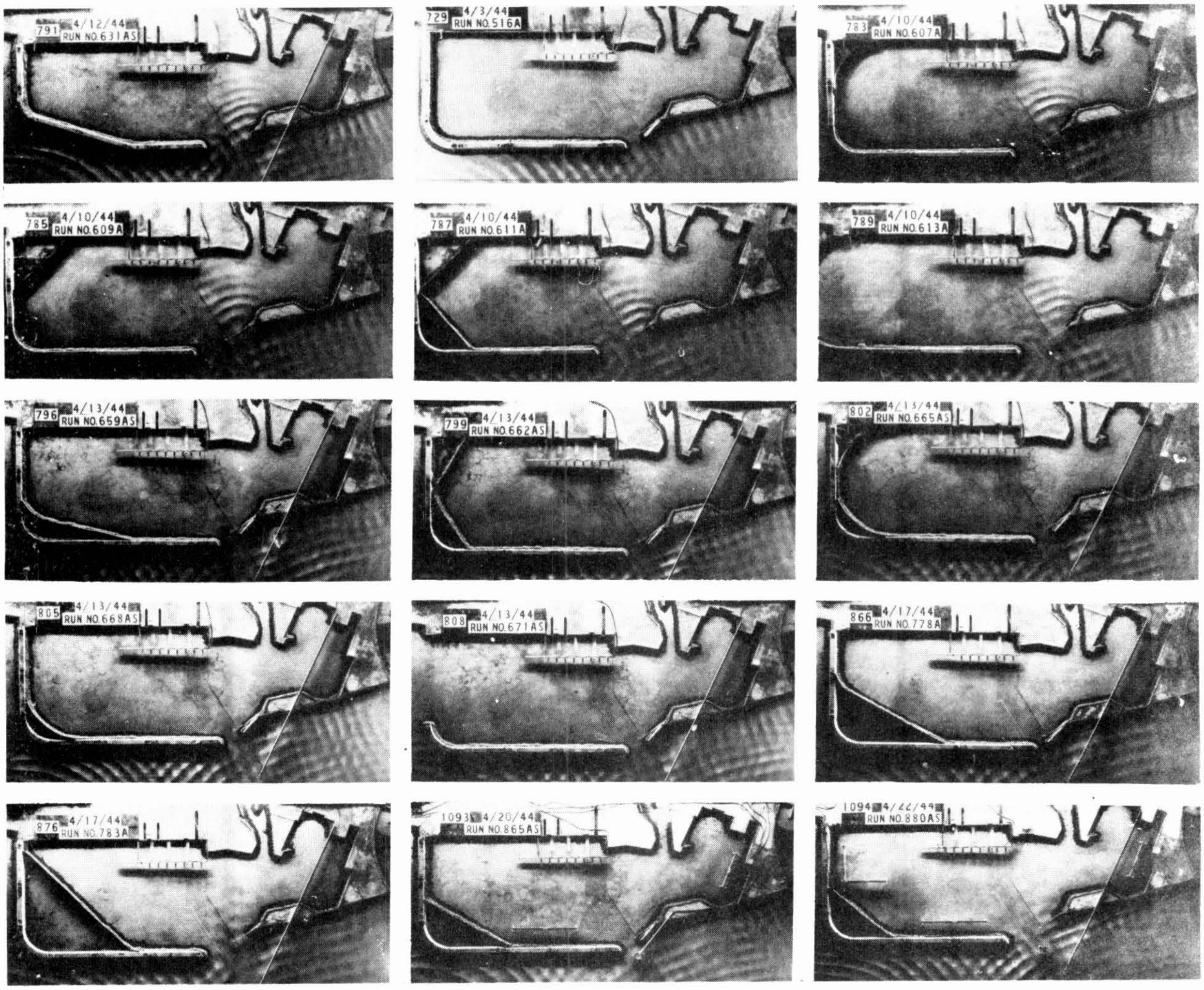
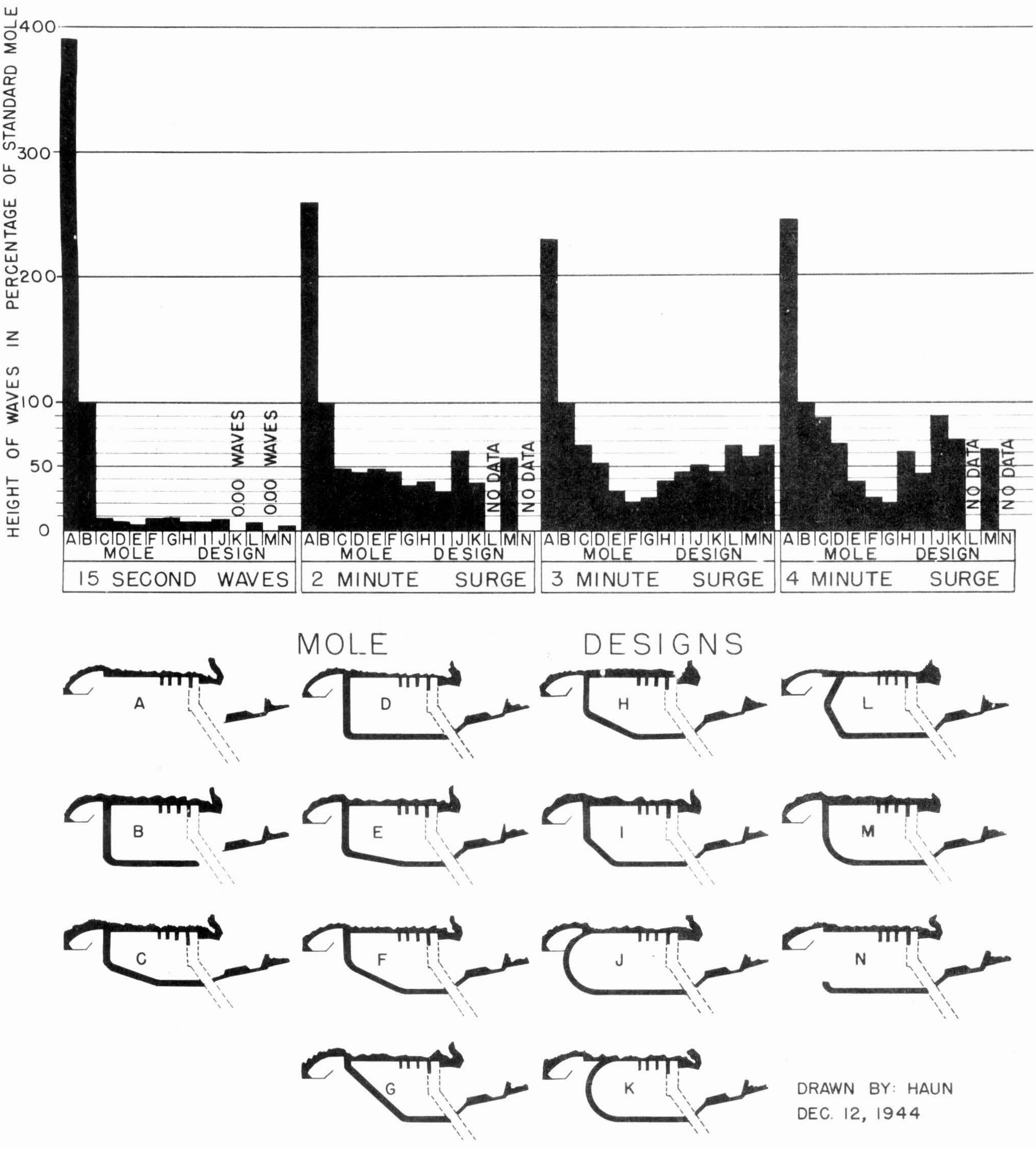


FIG. 69 WAVE PATTERNS OF 15 DIFFERENT MOLE CONFIGURATIONS WITH WAVES ORIGINATING FROM BOTH EAST AND WEST GATES



DRAWN BY: HAUN
DEC. 12, 1944

FIG. 70 COMPARISON OF AMPLITUDES FOR 14 MOLE CONFIGURATIONS
B = STANDARD DESIGN – 2070 FT. GATE
C = ORIGINAL DESIGN – BUT WITH GATE REDUCED TO 600 FT.

minute, three minute, and four minute surges. At the time of this study only eight elements were available for measuring vertical amplitudes. These were grouped in the area in front of the present drydocks and piers. Their location is shown by the photographs of Figure 69. Runs were also made on the model without mole to serve as a standard of comparison. The results are shown on the bar graph of Figure 70. The amplitude of motion in the critical area covered by the elements is presented as the ratio of the average movement recorded by all elements with a given mole configuration to the same average with the Standard mole configuration. It will be observed that the Standard mole modified with either a thirty degree (Figure 70H) or a forty-five degree (Figure 70G) diagonal in place of the square corner gives the lowest disturbance for the average of the different conditions and that the fifteen degree diagonal corner (Figure 70E) is also very good. Figure 71 shows the comparison of the effect of moles with these diagonals and with the original square corner. It will be seen from this presentation that there is little difference between the thirty degree and forty-five degree diagonal. Therefore, the thirty degree one was chosen as the configuration to be recommended since it cut off less of the basin. In making this recommendation, it was realized that the criterion of choice was the degree of disturbance in the vicinity of the drydocks and piers only and not the average disturbance in the basin as a whole. The reason for this was that this area had been defined by the Navy as the critical area which was to receive the best possible protection. In making this recommendation it was proposed that it be accepted as determining the construction only from the shore to the outer end of the thirty degree diagonal, since it was felt that it might be desirable to make some further experiments on slight changes in alignment of the final section from this point to the gate. This recommendation was accepted and construction was commenced in the harbor on this basis. Therefore, the remaining studies were all made with mole configurations in accordance with it.

(2) Final comparison of 90° and 30° mole corners. At a later date when the complete set of wave measuring elements were available for use in the basin, measurements were made of the relative performance of the Standard Design with the square corner and modified with the thirty degree diagonal. Their characteristics were compared with each other and with the behaviour of the basin area with no mole in place. The results are presented in contour maps of vertical amplitude of motion. Figures 72 and 73 show the behaviour of the standard mole as compared to the unmodified basin. The first map shows motion due to the fifteen second wave train and the second one the three minute surge. Figures 74 and 75 show the corresponding information for the thirty degree diagonal modification and, finally, Figures 76 and 77 show the direct comparison of the two moles. It will be noted that in all these comparisons the mole gate opening is 600 ft. toe to toe of slope, which gives a width of 750 ft. at the water level. Figure 77 for the three minute surge train shows clearly the superiority of the thirty degree diagonal in reducing the vertical motion along the shore of the drydocks area. It will be noted that

the disturbance in the channel directly inside the gate is slightly greater for the diagonal mole. The absolute magnitude of this disturbance is small and it is felt that it will have little effect on the difficulty of navigation in this area.

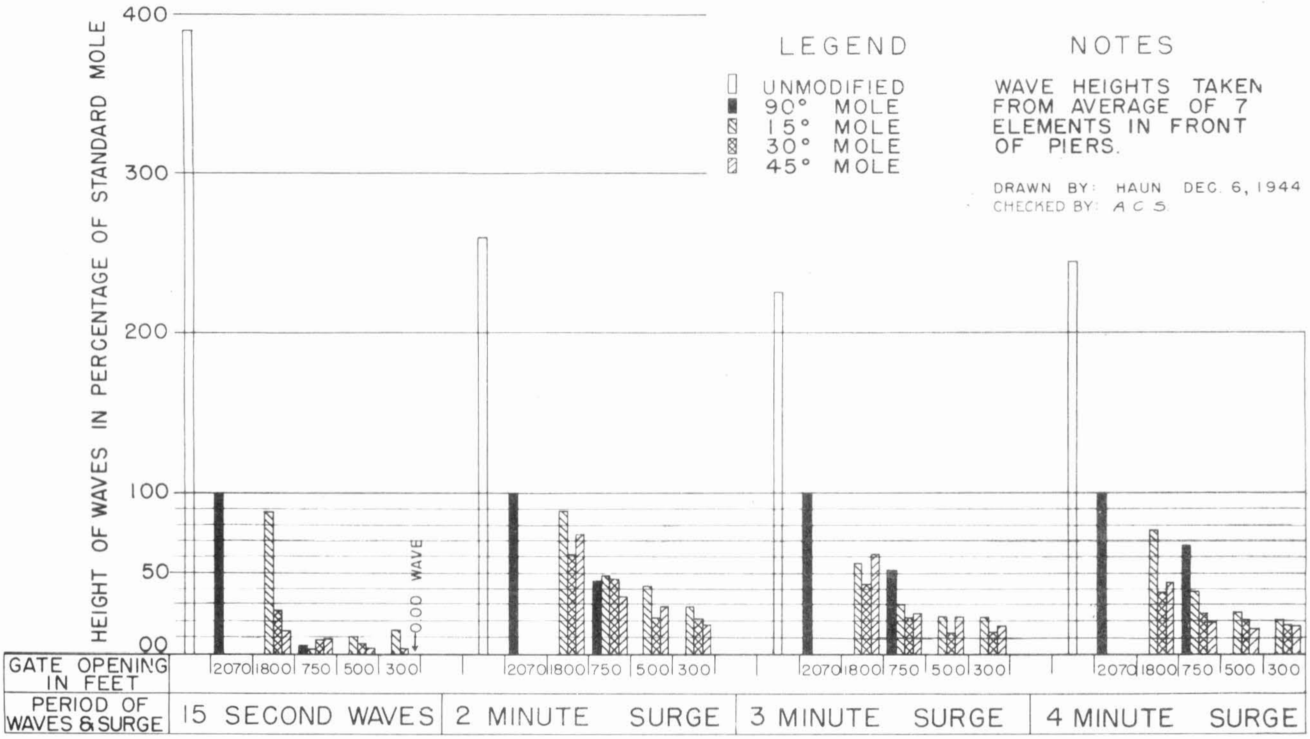


FIG. 71 VERTICAL AMPLITUDES AT DRYDOCK AREA FOR MOLES WITH 15^{0}, 30^{0}, 45^{0} DIAGONAL CORNERS COMPARED WITH THOSE FOR ORIGINAL 90^{0} CORNER DESIGN

(d) Gate opening. The location of the mole gate was investigated on the first model and the conclusion reached that it should remain at the location of the 45 ft. navigation channel from the east gate to the drydock area. However, the width of the opening was left undetermined. For the purposes of navigation, it is desirable to have the maximum gate opening consistent with the desired reduction of the disturbance within the basin. Three groups of tests were run to investigate the effect of gate opening on this model, i.e., (1) with an opening of 2070 ft. from toe to toe of slope symmetrically spaced about the 750 ft. wide navigation channel; (2) a 1320 ft. opening which was obtained by reducing the 2070 ft. opening by installing an extension to Pier A out to the line of the navigation channel; (3) a 600 ft. opening formed by extending the mole until it reached the navigation channel. A series of runs was made with these three configurations with (a) 15 second waves; (b) 3 minute surges; (c) 6 minute surges. The configuration of the gate area is shown in photographs, Figures 78, 79 and 80, and the contour maps of the vertical movements for the three different lengths of wave trains

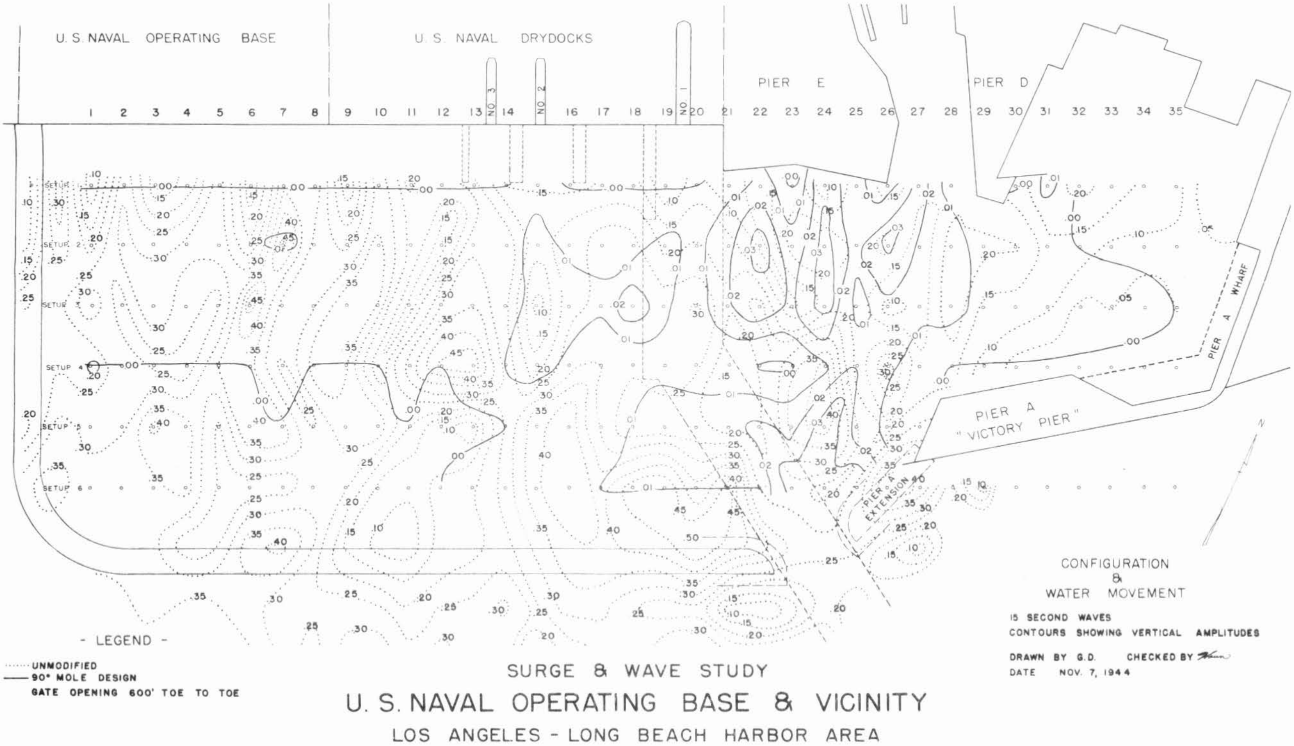


FIG. 72 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
STANDARD MOLE VS. UNMODIFIED BASIN

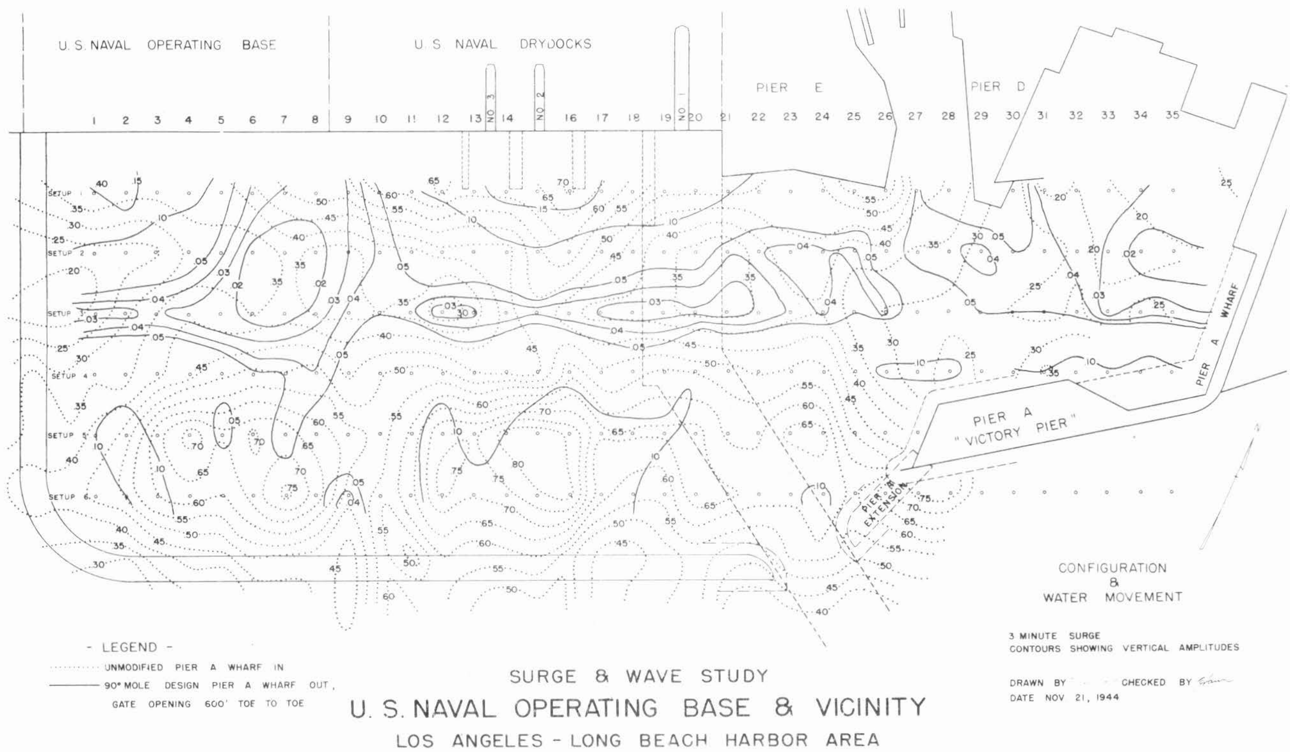


FIG. 73 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
STANDARD MOLE VS. UNMODIFIED BASIN

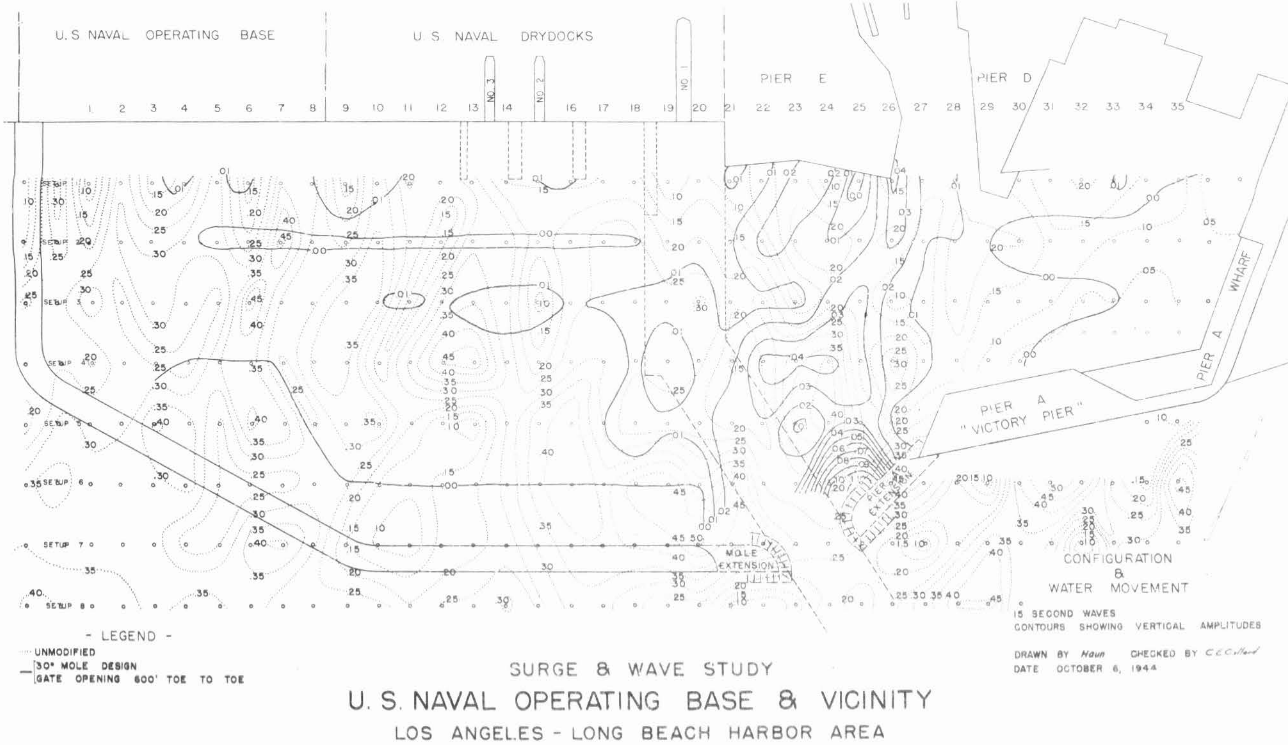


FIG. 74 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
30° DIAGONAL MOLE VS. UNMODIFIED BASIN

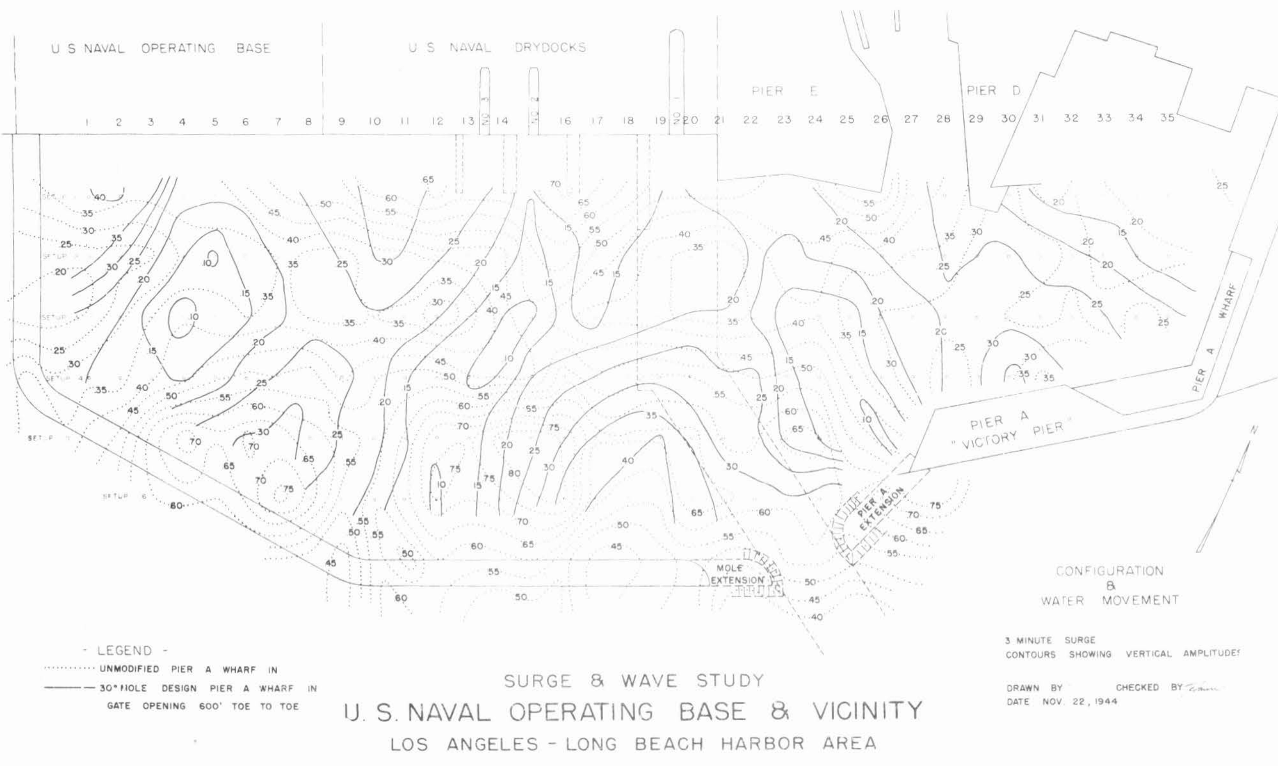


FIG. 75 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
30° DIAGONAL MOLE VS. UNMODIFIED BASIN

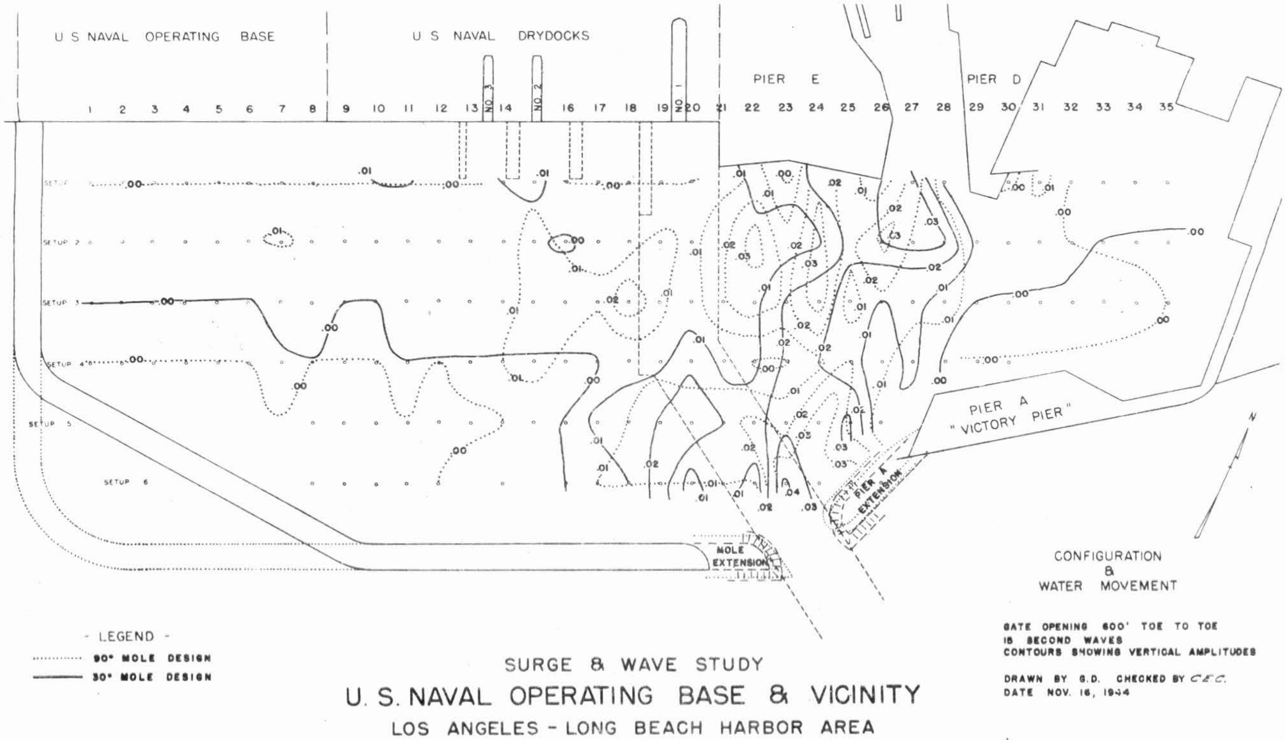


FIG. 76 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
30° DIAGONAL MOLE VS. STANDARD 90° MOLE

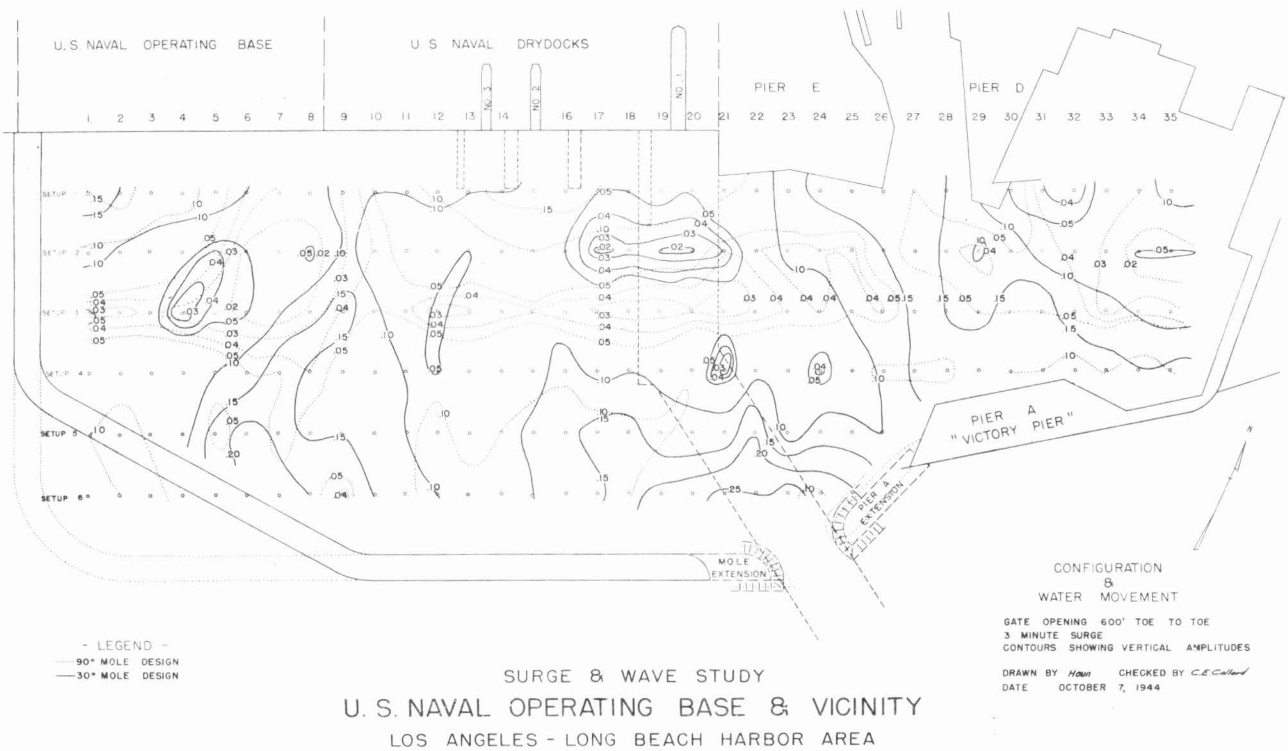


FIG. 77 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
30° DIAGONAL MOLE VS. STANDARD 90° MOLE

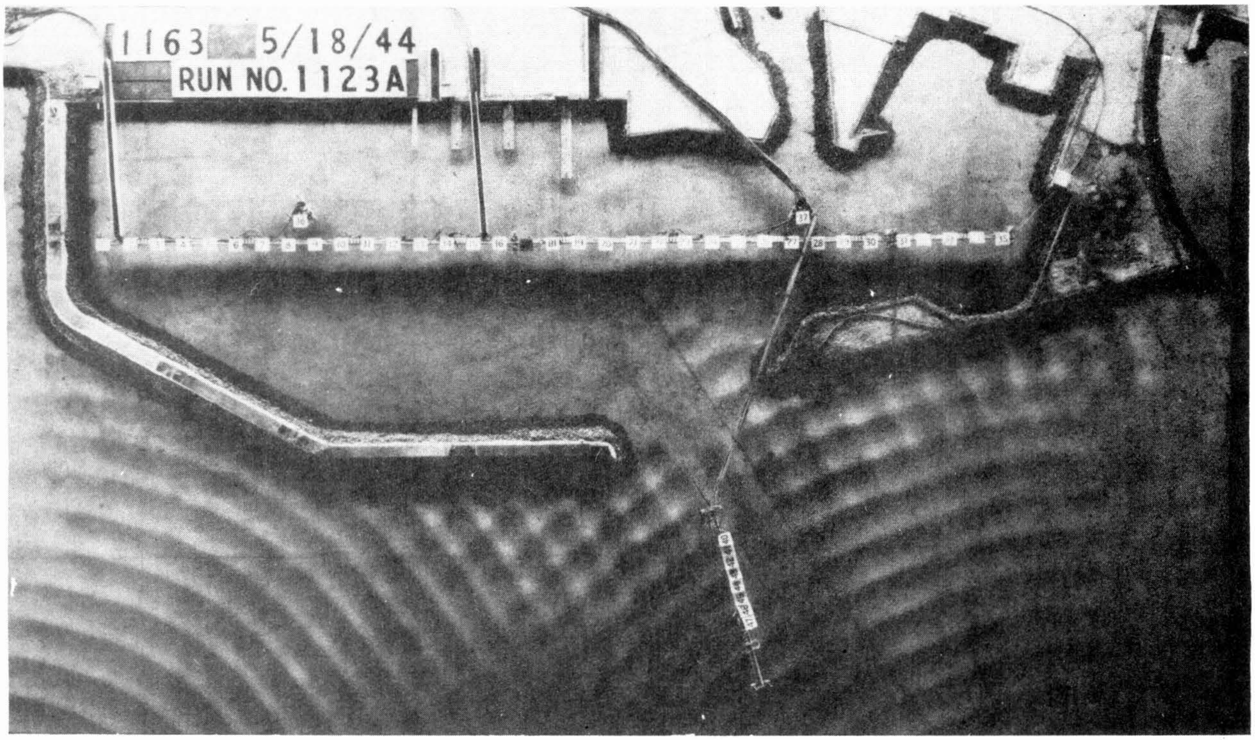


FIG. 78 30° MOLE - 2070 FT. GATE OPENING

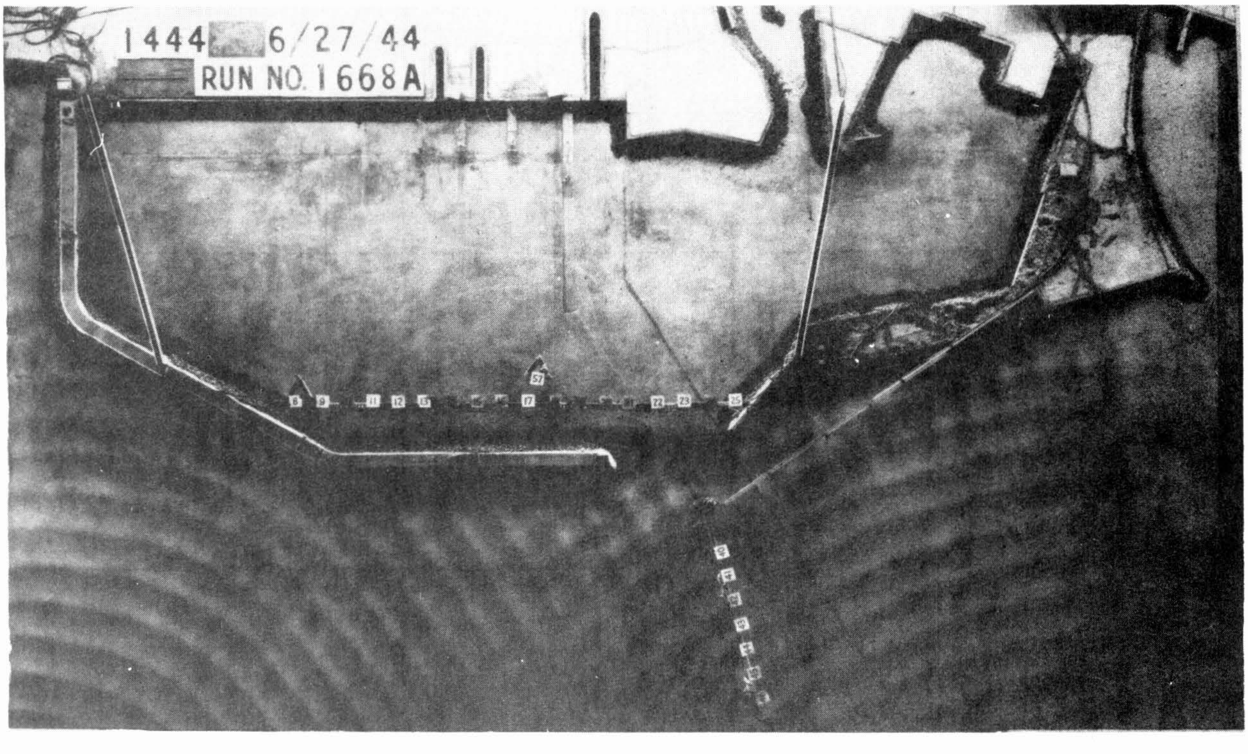


FIG. 79 30° MOLE - 1320 GATE OPENING

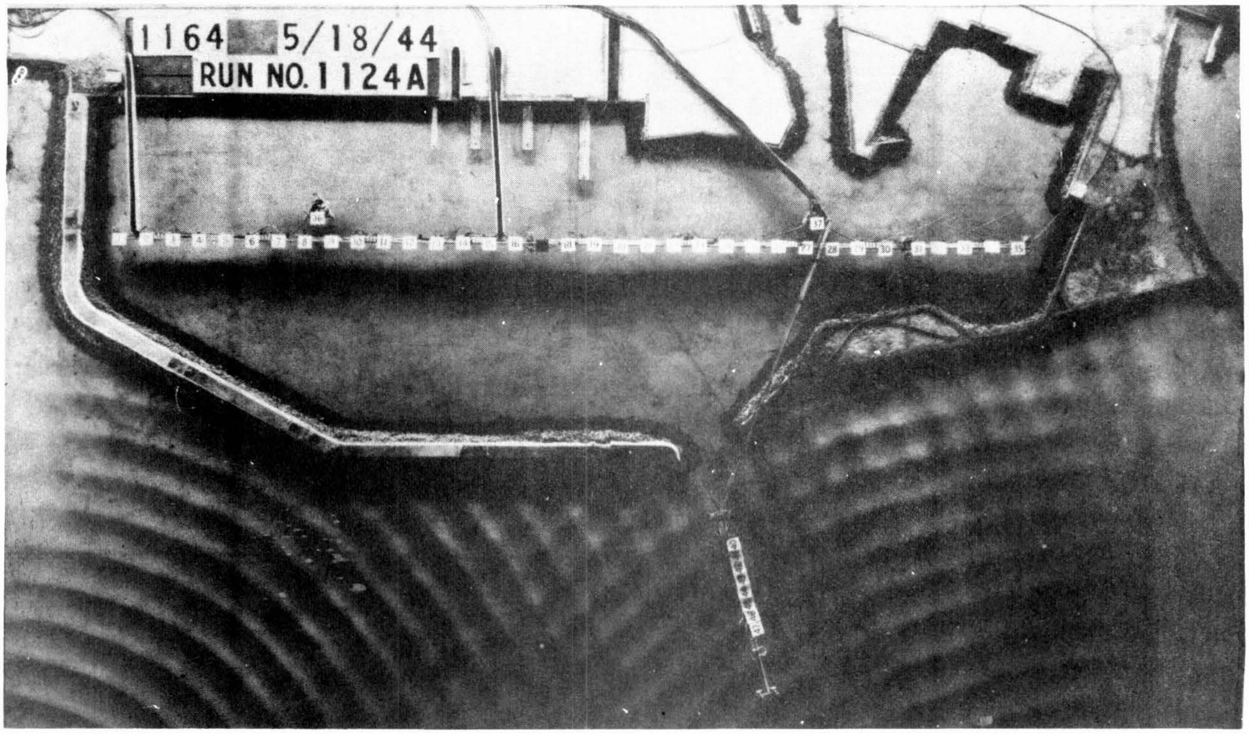


FIG. 80 30° MOLE - 600 FT. GATE OPENING

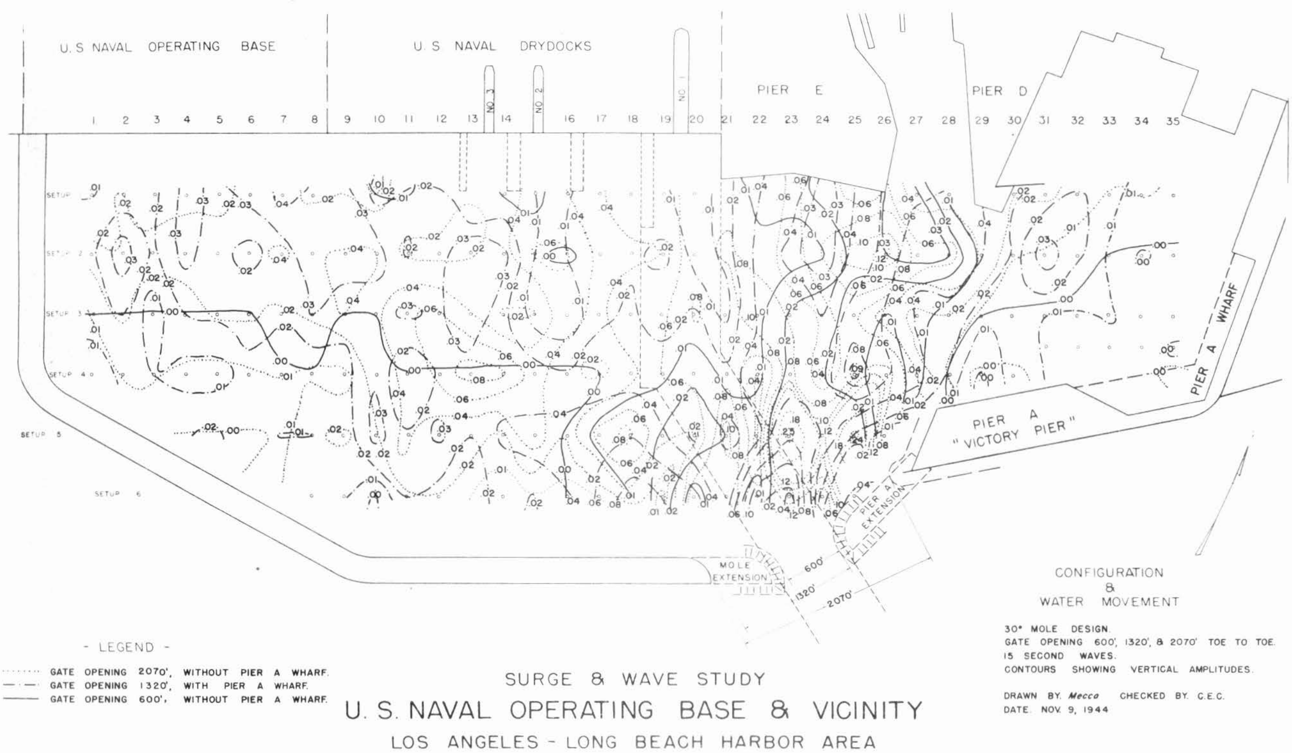


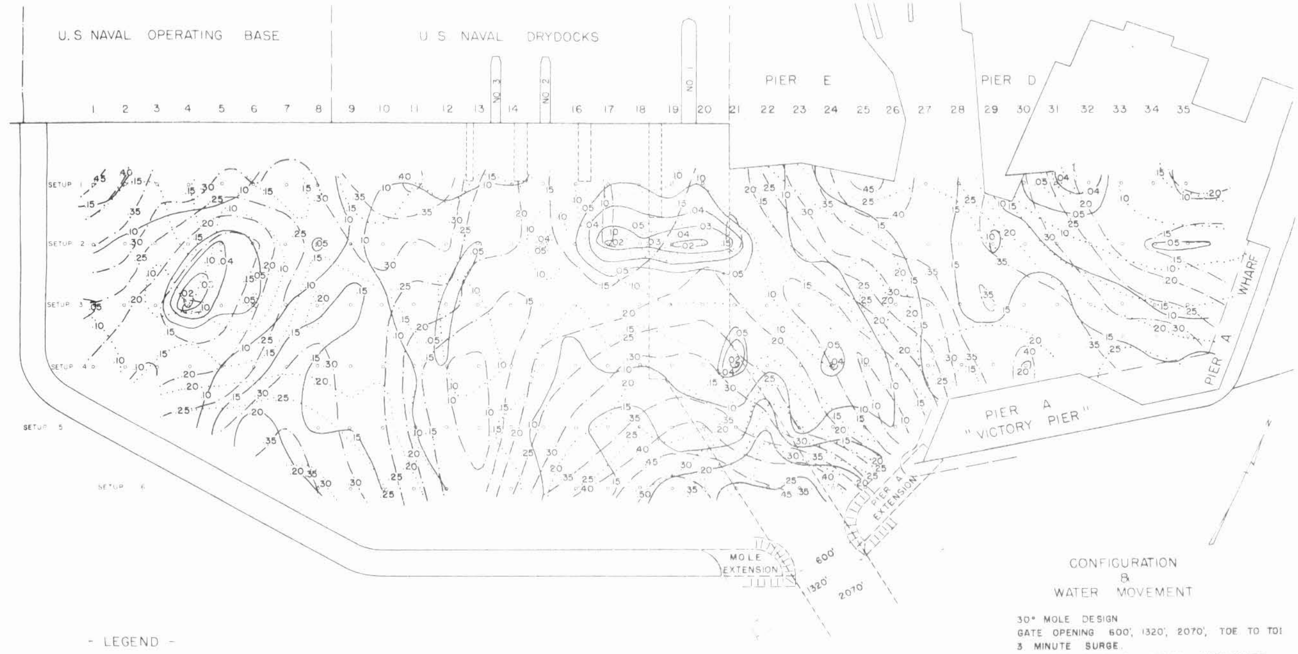
FIG. 81 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
600 FT., 1320 FT. AND 2070 FT. GATE OPENINGS

are shown in Figures 81, 82 and 84. In all cases, it will be noted that there is an improvement in the conditions in the basin as the gate opening is made smaller and that the improvement is more marked with the reduction from 1320 ft. to 600 ft. than from 2070 ft. to 1320 ft. However, the six minute surge conditions are far from satisfactory even with the smallest gate opening. The same general trend is seen in Figure 71 (page 97) which was used in the preceding section to present the results of the different mole configurations.

(e) Effect of additional structures within mole. One of the difficulties encountered in carrying out this entire study arose from the fact that conditions both in Long Beach harbor and in the area of the drydocks were very fluid and that several unforeseen major changes in the construction within this area took place after the study commenced. Consequently, a series of tests was made on this model to determine the effect of the installation of different structures that were proposed or authorized and, in some cases, actually under construction.

(1) Pier A wharf. The first of these structures to be investigated was what is known as "Pier A Wharf", an extension to Pier A or Victory Pier of the Long Beach Harbor. Figures 84 and 85 show the model with and without this construction. Figures 86 and 87 are the water movement contour maps showing the comparative behaviour of the basin with and without the wharf. It will be noted that both for the fifteen second and the three minute wave trains, the water movement is appreciably increased by the presence of Pier A Wharf. This increase is particularly noticeable in the Long Beach harbor area, although the effect of it is appreciable throughout the entire basin. The reason for this is that it removes some corners and probably eliminates some wave reflections that produced both interference and damping and hence, decreased the disturbance. Thus, the evidence is that Pier A Wharf is a detriment to the conditions both within Long Beach harbor and in the vicinity of the Naval Base. No runs are available from Model 2 to confirm this conclusion for conditions as they were with the mole removed. However, comparative runs were made on the Model 3 basin without the mole in place to ascertain the effect of Pier A wharf under these conditions. Figure 88 shows the results from the fifteen second wave trains. It will be seen that again Pier A wharf increases the vertical amplitude of the motion both in Long Beach harbor and in the drydocks area. In these tests, however, the effects in the drydock area are not so evident, probably because they are masked by the great increase in amplitude of the disturbance in this area without the mole. Since, at the time these tests were completed, the actual construction in the harbor was well under way, it was decided to continue the studies with Pier A wharf as a permanent part of the model.

(2) Pier E. Another new piece of construction proposed by the City of Long Beach was a large extension to Pier E. Figures 89 and 90 show the vertical amplitude contour maps for the fifteen second and three minute surges for the model with and without this



30° MOLE DESIGN
GATE OPENING 600', 1320', 2070', TOE TO TOE
3 MINUTE SURGE
CONTOURS SHOWING VERTICAL AMPLITUDES

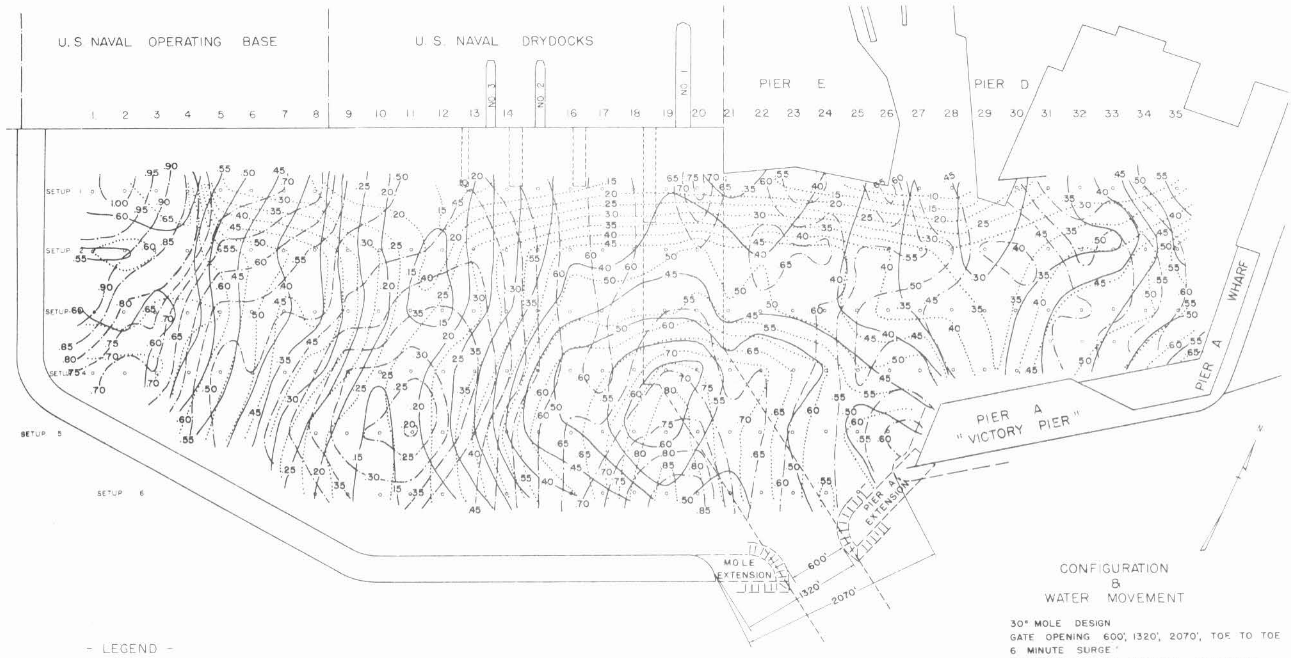
DRAWN BY Mecco CHECKED BY
DATE NOV 14, 1944

- LEGEND -

GATE OPENING 2070', WITHOUT PIER A WHARF
GATE OPENING 1320', WITH PIER A WHARF
GATE OPENING 600', WITHOUT PIER A WHARF

SURGE & WAVE STUDY
U.S. NAVAL OPERATING BASE & VICINITY
LOS ANGELES - LONG BEACH HARBOR AREA

FIG. 82 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
600 FT., 1320 FT. AND 2070 FT. GATE OPENING



30° MOLE DESIGN
GATE OPENING 600', 1320', 2070', TOE TO TOE
6 MINUTE SURGE
CONTOURS SHOWING VERTICAL AMPLITUDES

DRAWN BY CHECKED BY
DATE NOV 16, 1944

- LEGEND -

GATE OPENING 2070' WITHOUT PIER A WHARF
GATE OPENING 1320' WITH PIER A WHARF
GATE OPENING 600' WITHOUT PIER A WHARF

SURGE & WAVE STUDY
U.S. NAVAL OPERATING BASE & VICINITY
LOS ANGELES - LONG BEACH HARBOR AREA

FIG. 83 VERTICAL MOVEMENT CAUSED BY 6 MINUTE SURGE
600 FT., 1320 FT., AND 2070 FT. GATE OPENING

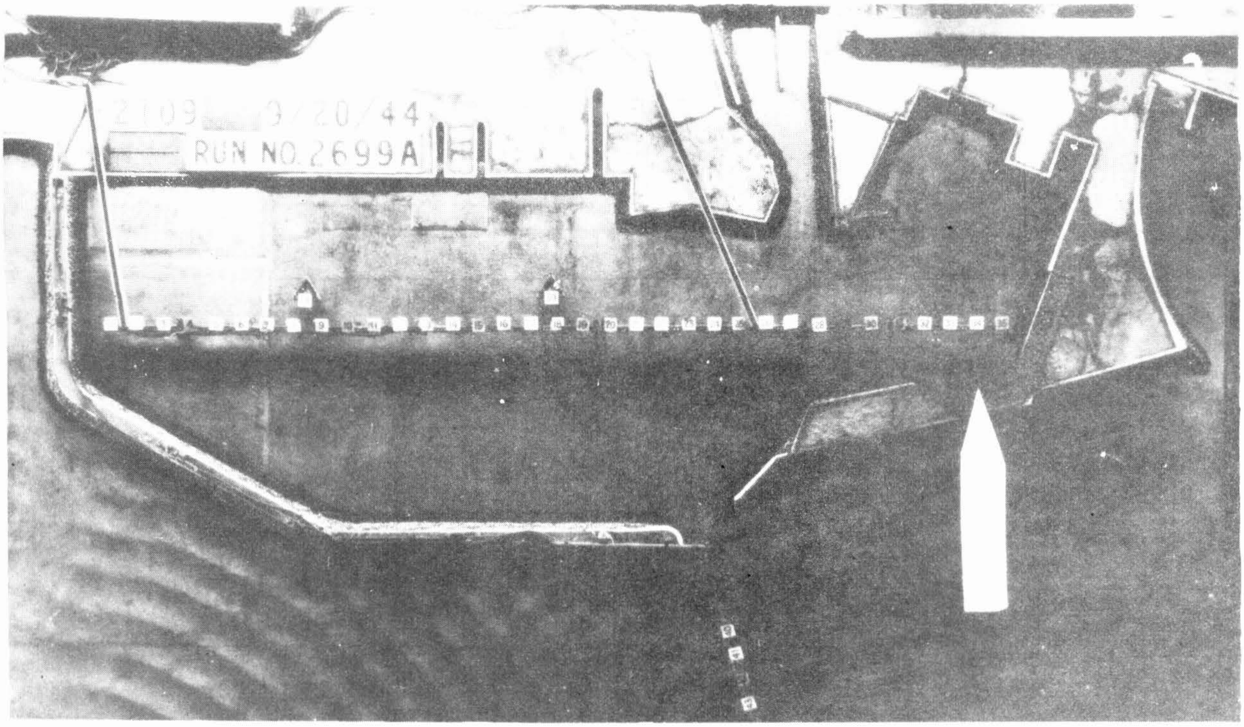


FIG. 84 30° MOLE WITH PIER A WHARF

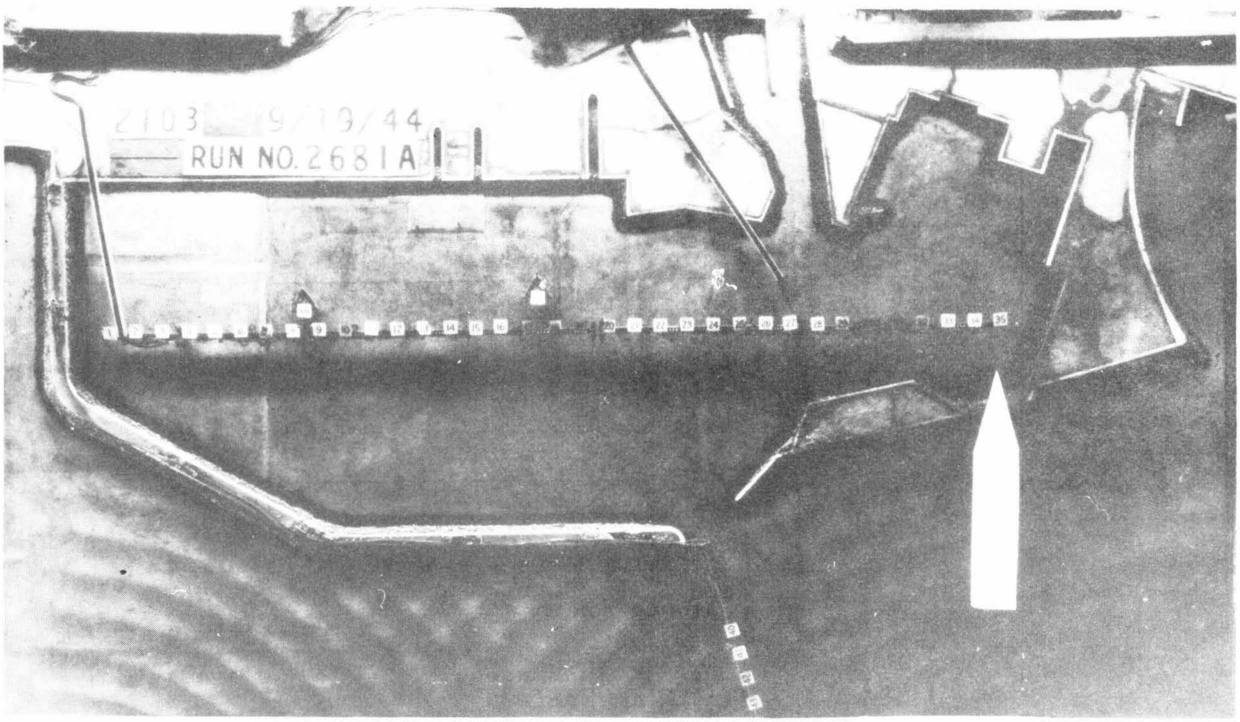
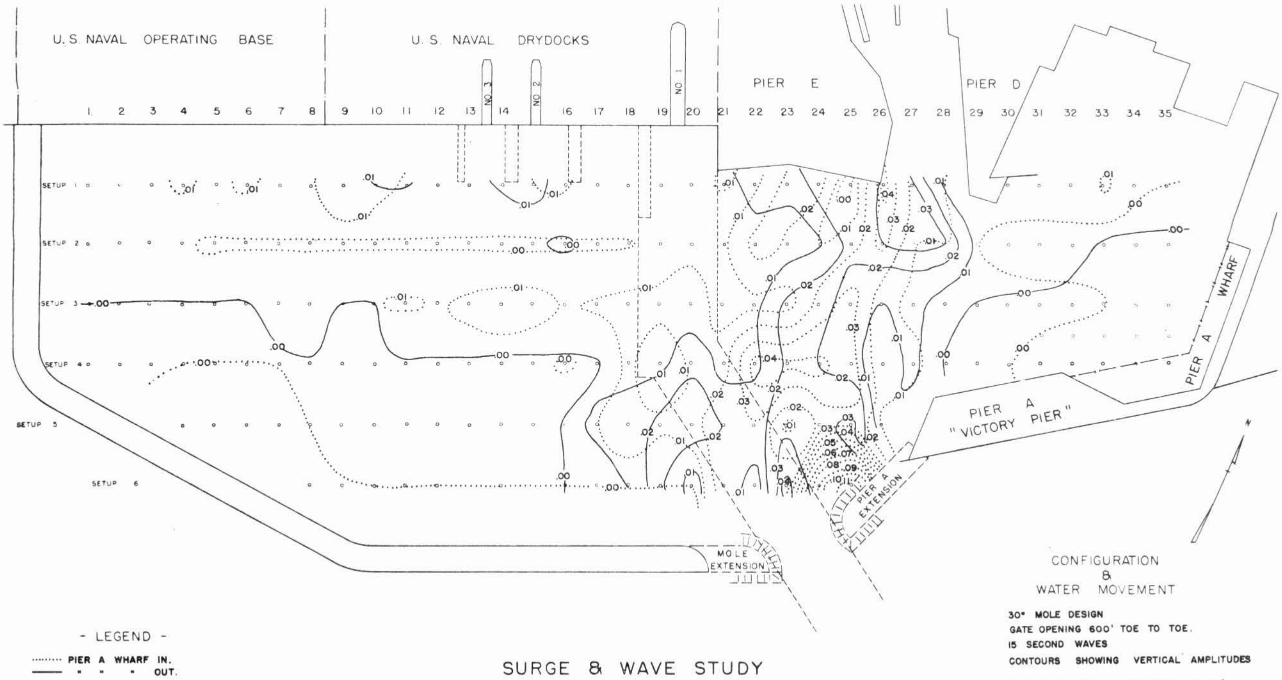
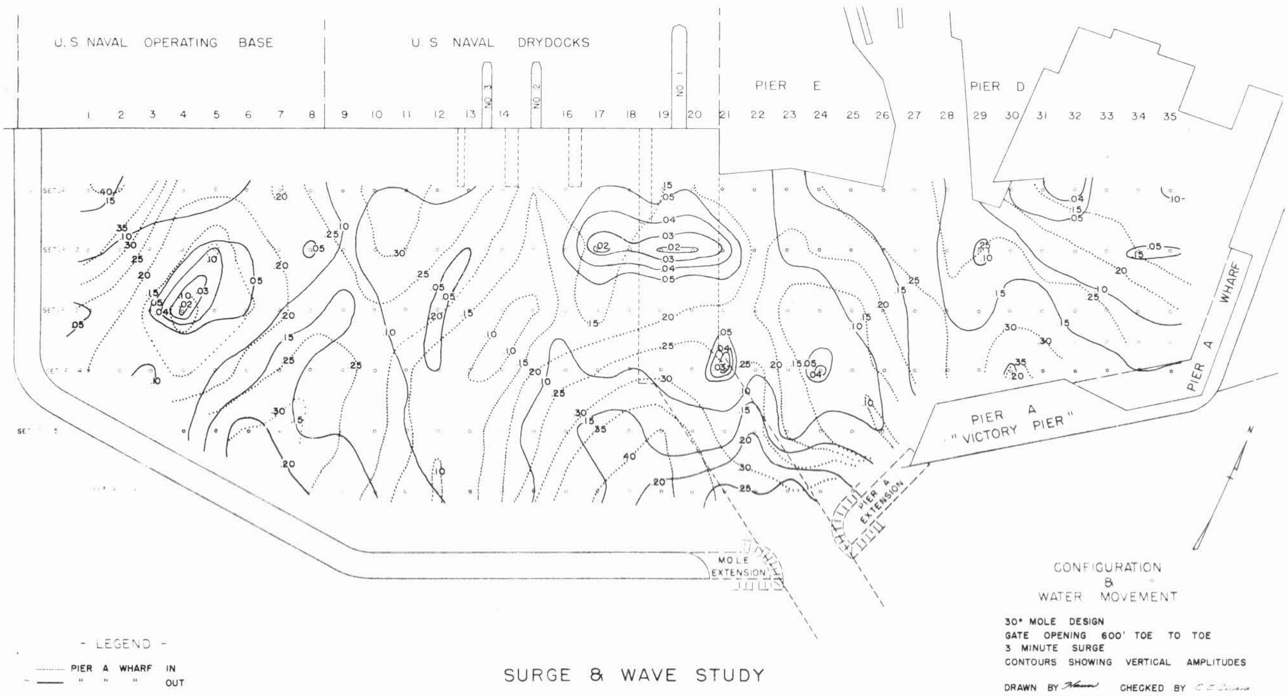


FIG. 85 30° MOLE WITHOUT PIER A WHARF



SURGE & WAVE STUDY
U. S. NAVAL OPERATING BASE & VICINITY
LOS ANGELES - LONG BEACH HARBOR AREA

FIG. 86 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES WITH AND WITHOUT PIER A WHARF



SURGE & WAVE STUDY
U. S. NAVAL OPERATING BASE & VICINITY
LOS ANGELES - LONG BEACH HARBOR AREA

FIG. 87 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT PIER A WHARF

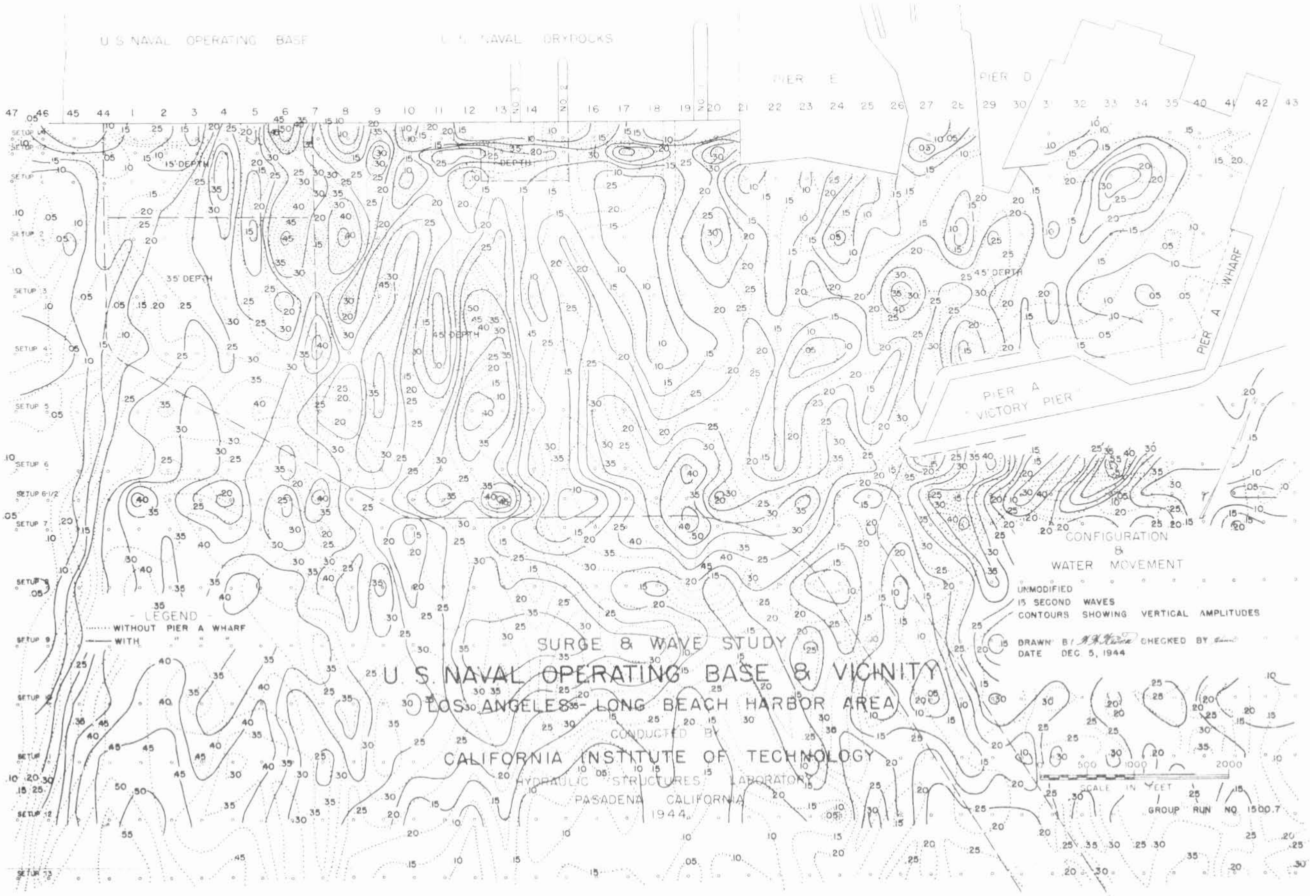


FIG. 88 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES WITH AND WITHOUT PIER A WHARF — UNMODIFIED BASIN

extension in place. It will be observed in both cases that the presence of Pier E extension produces a slight decrease in the amplitude, probably due to an increase in the basin damping and the introduction of an additional group of interfering wave reflections.

(3) Drydock, marginal wharf and mole piers. Another structure proposed for the future development of the drydock area was the installation of a large fill for the construction of a new drydock beginning at Pier 4 and running to the west. Figure 91 gives a comparison of the height of 3 minute waves with and without this drydock fill. Figure 92 shows the effect on the three minute waves of the further addition of marginal wharf and mole piers to the outer arm of the mole. It will be observed that the general result of all of these additions is a continuing decrease in the amplitude of the motion within the basin, again due probably to increased damping.

(4) Effect of moving mole piers. In Figure 92, however, it will be observed that the amplitude of motion at the ends of the mole piers is rather high. In an effort to reduce it, the piers were moved westward until they lay along the thirty degree diagonal

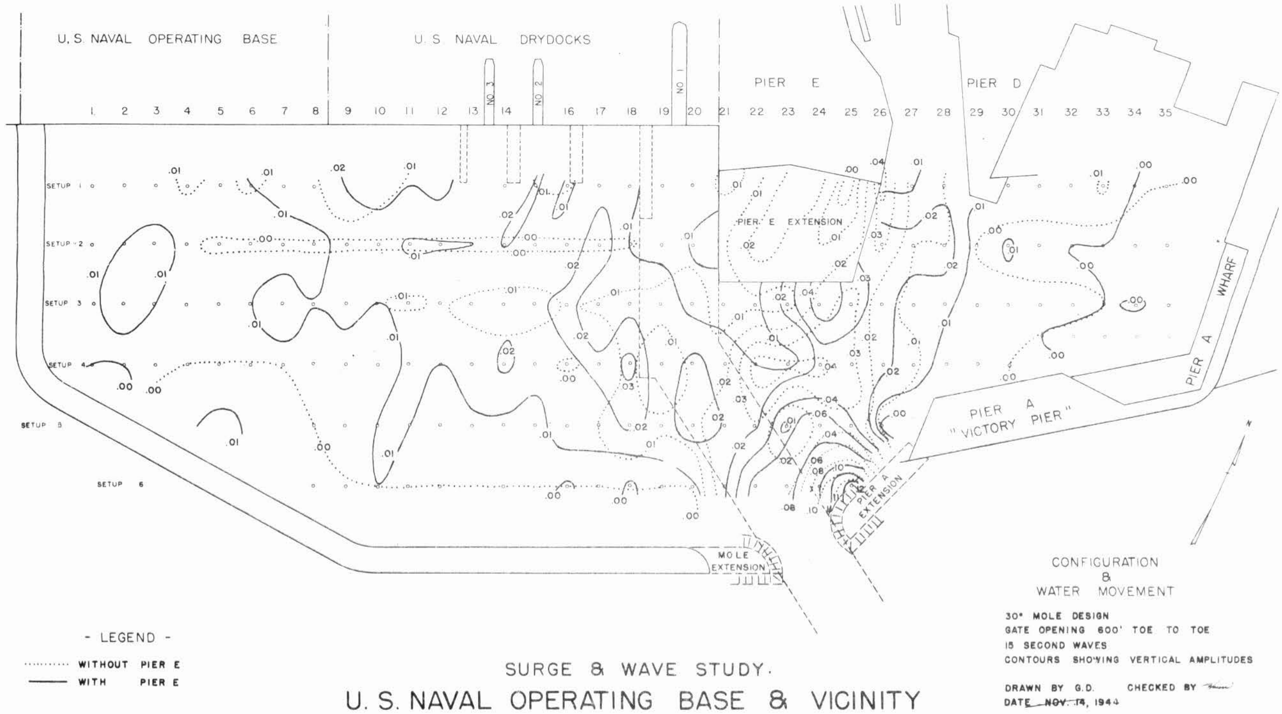


FIG. 89 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES WITH AND WITHOUT PIER E EXTENSION

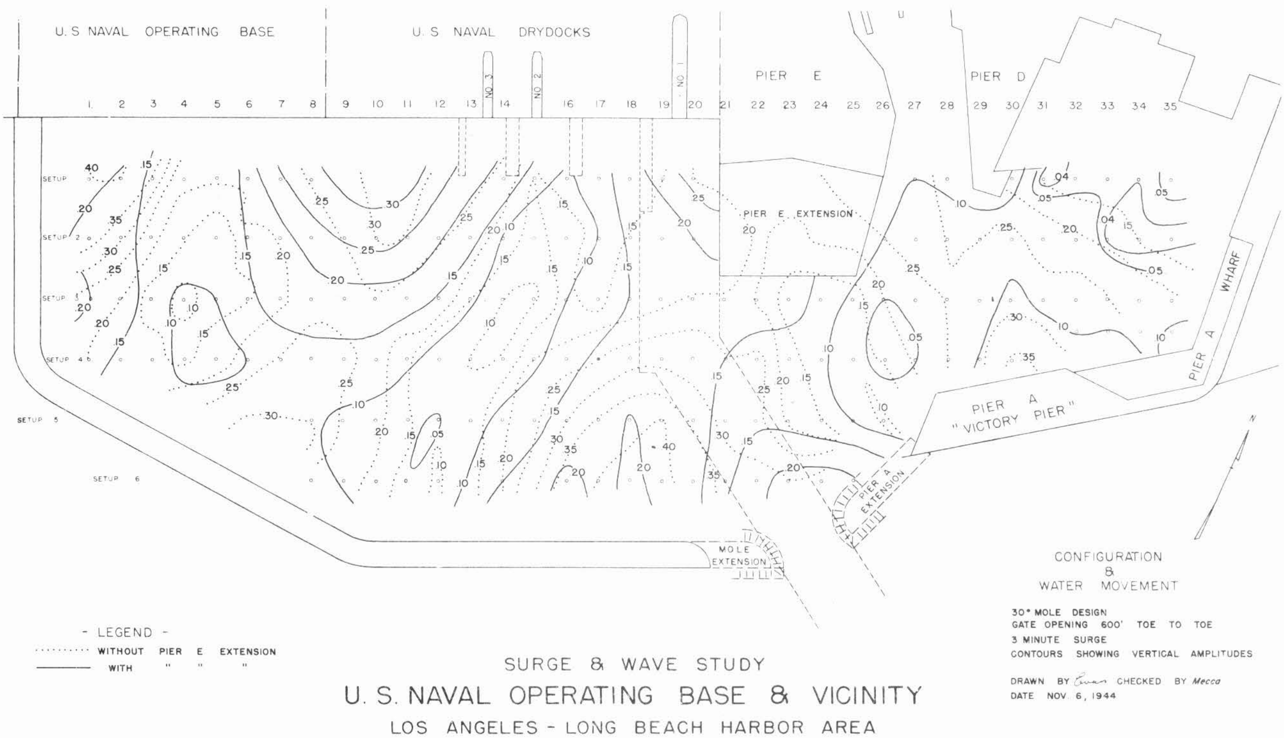


FIG. 90 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT PIER E EXTENSION

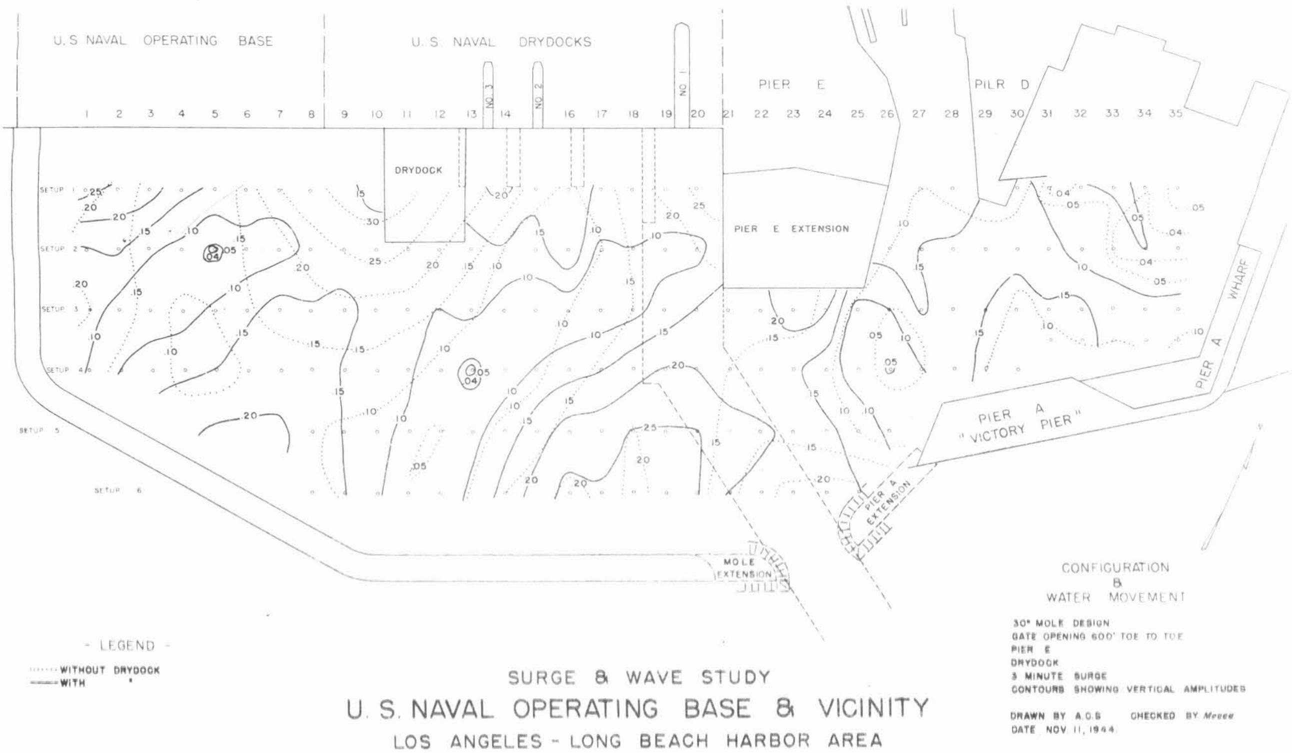


FIG. 91 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT DRYDOCK

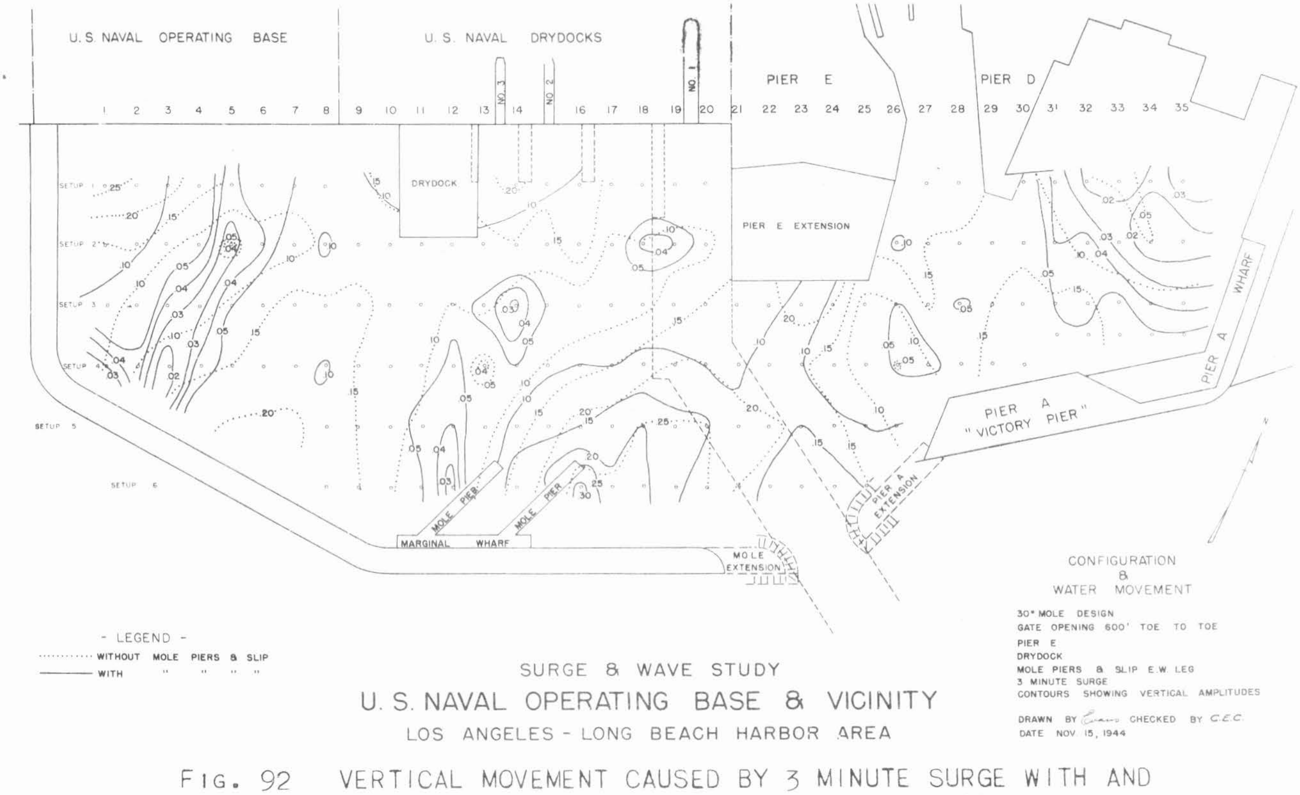


FIG. 92 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT MARGINAL WHARF AND MOLE PIERS

portion of the mole. Figure 93 shows that this is a more satisfactory position but that the movement still is rather large. A solid or opaque protective wharf was then added in the original position on the parallel leg of the mole. The results are shown in Figure 94. It will be seen that the vertical motion in the vicinity of the mole piers has now been reduced to a very satisfactory amount. It may have been noted that in making these comparisons, the results have been presented for the three minute surge train only. This is because the amplitude of the motion caused by the fifteen second waves with these configurations was so small as to be insensitive to these small changes. Furthermore, the conditions due to the three minute surge train are felt to be a better measure of the degree of excellence of the basin.

(5) Effect of opaque piers. Visual observation of ship models docked at Piers 1, 2 and 3 indicated that even though the presence of the mole had reduced the motion within the basin to a considerable degree, the ship models still had quite large excursions under the effects of three minute or six minute surges. In an endeavor to reduce or eliminate the cause of these movements, tests were made with solid or opaque piers in place of the existing pile or transparent Piers 1, 2 and 3. Figure 95 shows a comparison of the wave heights obtained with and without the solid piers. It will be noted that docking conditions are markedly improved as the motion of the water under the piers is eliminated.

(6) Effect of mole and basin structures on Long Beach harbor. All of the preceding maps show clearly one point of interest to Long Beach Harbor, i.e., that the presence of the mole, both with or without any of the accompanying internal structures, always improves conditions in Long Beach Harbor, and this improvement is more than enough to counterbalance the bad effects of the construction of Pier A wharf.

(f) Horizontal water motions. A series of reflector shots obtained as described in Section VII-A were made for both 600 ft. and 2070 ft. gate openings. This series covered the various basin structures described in this section. The traces of the reflectors indicate the horizontal motion. Figure 96 shows the effects of fifteen second, three minute, and six minute wave trains with the 2070 ft. gate and Figure 97 shows the same group of configurations with the 600 ft. gate. It will be observed that these reflector shots, which delineate the horizontal water motion, confirm the results obtained by measuring the vertical amplitudes of the water motion. The decrease in motion which accompanies the decrease in gate width is clearly visible both for the fifteen second and the three minute wave trains. The damping effect of the addition of the various structures to the basin shoreline can be clearly traced. However, this series of reflector shots has been superseded by a later study of approximately the same scope which was made on Model 3 with the final mole and basin design. Therefore, the more complete consideration of the horizontal water motion under this range of conditions will be found in the section on Model 3.

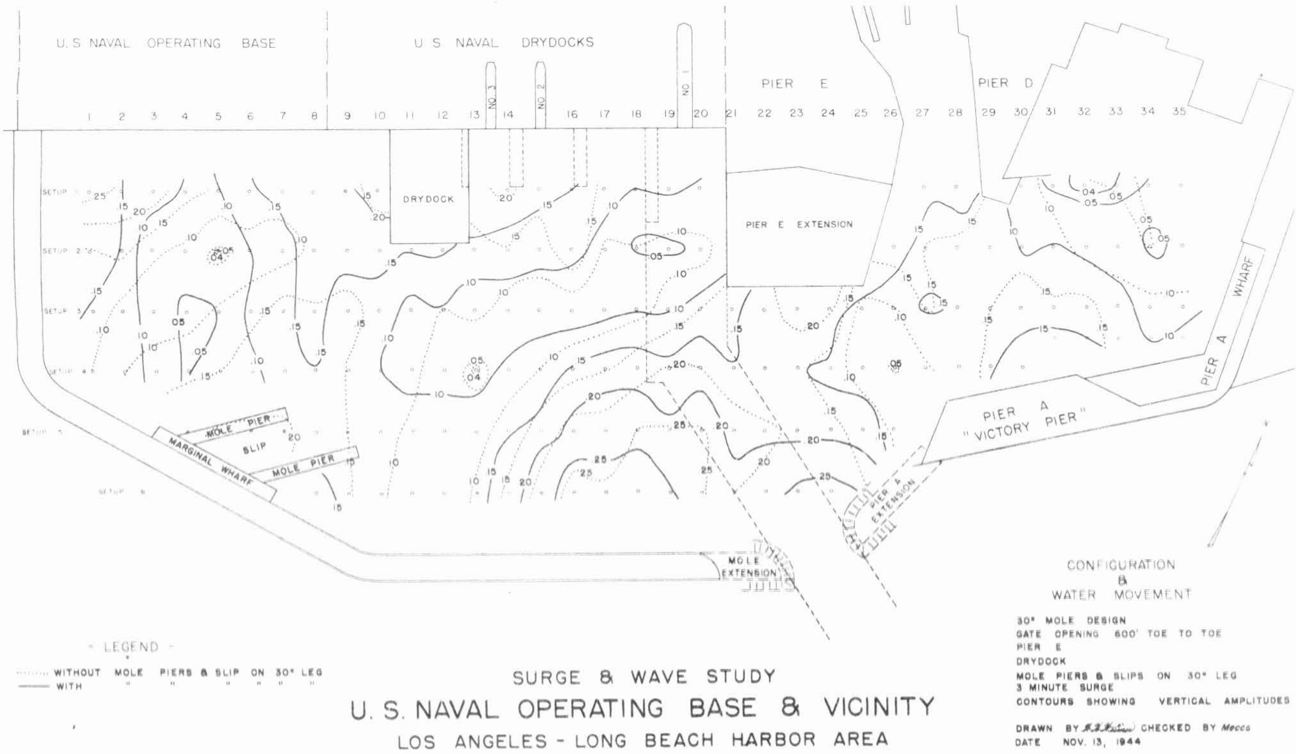


FIG. 93 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT MARGINAL WHARF AND MOLE PIERS ON 30° DIAGONAL

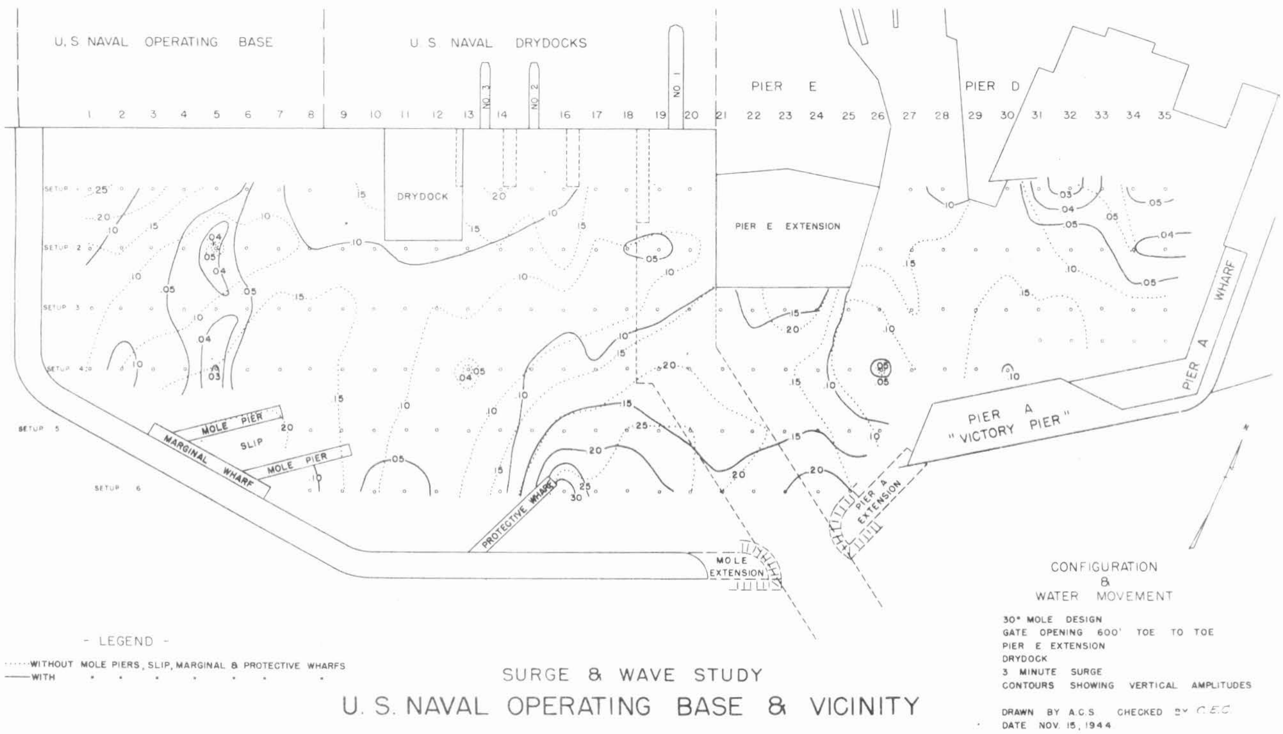


FIG. 94 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT MARGINAL WHARF, MOLE PIERS AND OPAQUE PROTECTIVE WHARF

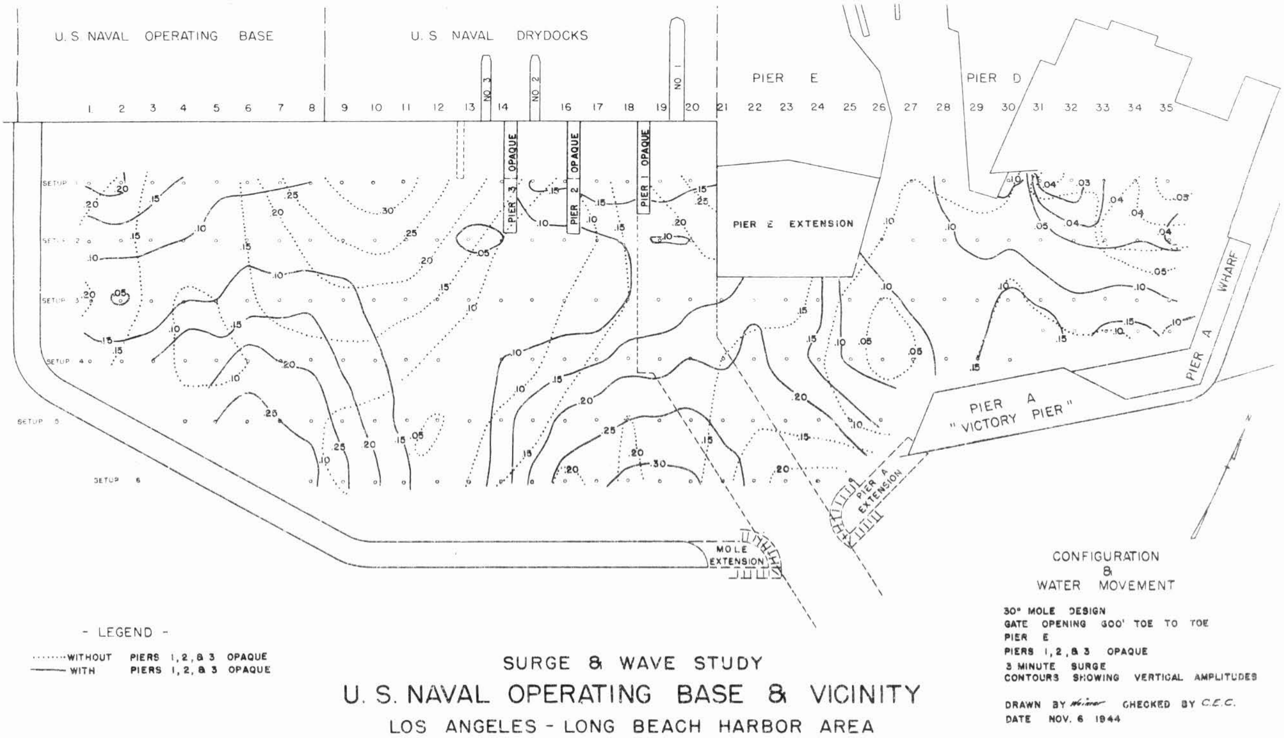


FIG. 95 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT OPAQUE PIERS 1, 2 AND 3

In interpreting these photographs it will be noted that the reflectors show a steady drift as well as oscillatory motions. The drift motion was not completely reproducible, indicating that more than one circulation pattern was possible for given conditions of mole configurations and waves. However, the oscillatory paths were reproducible and can, therefore, be relied upon to give the horizontal motions of the water.

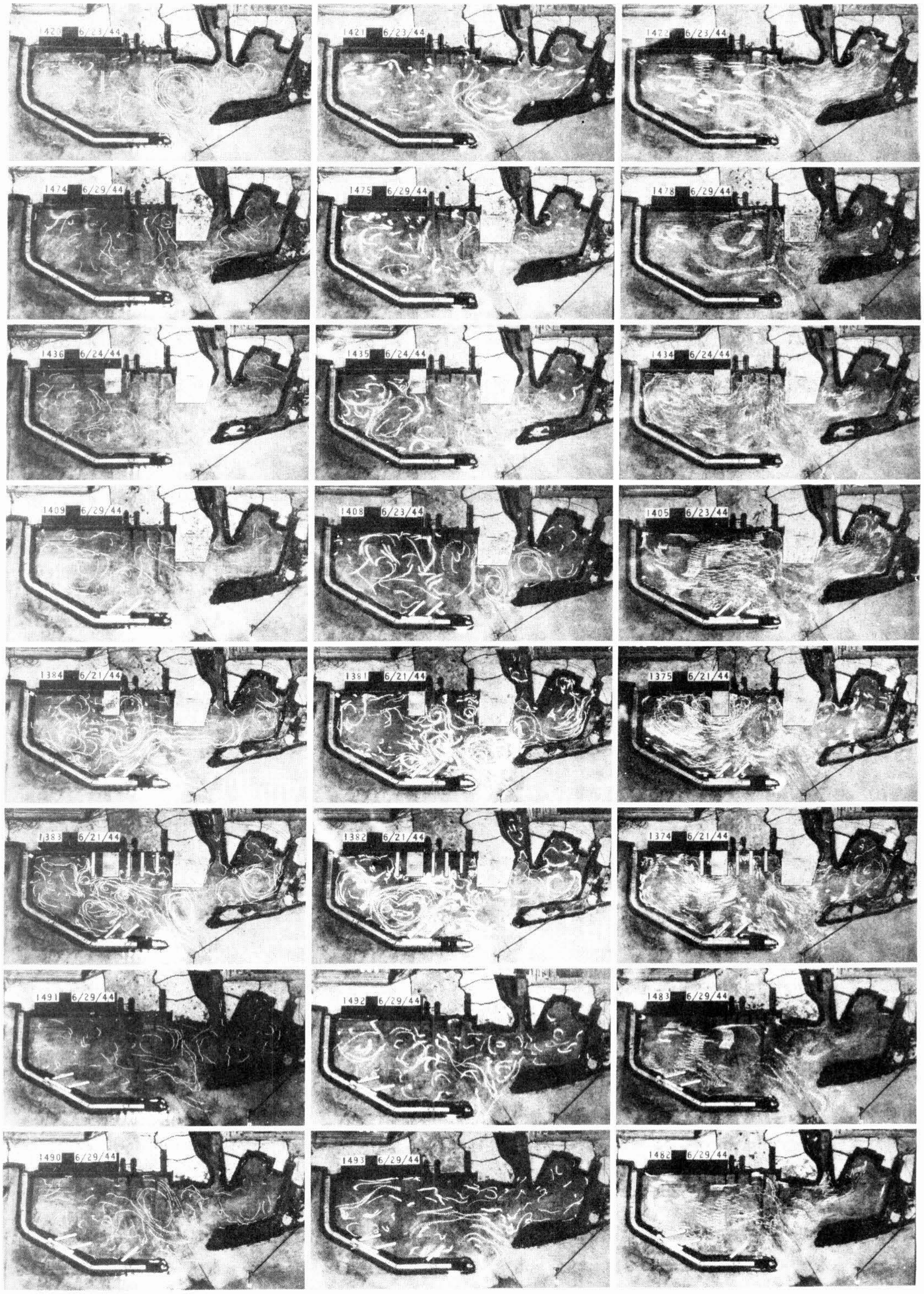
(g) Shape of mole ends. Near the termination of the investigation on Model 2, a proposal was made to increase the width of the mole from 287.5 ft. to 537.5 ft. It will be seen that this increase was sufficient to make it necessary to consider the shape of the mole ends, since the width of the mole was no longer small as compared to the wave length. At the same time it was proposed by the Naval Operating Base that the end of the Pier A extension be moved seaward to clear the line of the sub-channel from the main channel into Long Beach harbor. A series of tests was made with this new mole alignment and with two forms of mole ends - first, the semi-circular ends proposed by the Naval Operating Base and second, the pointed ends formed by extending the mole to the line of the channel and cutting them off at a diagonal along the edge of the channel. These two end-shapes, together with the results of the vertical amplitude studies, are seen in Figures 98 and 99. It will be observed that for both wave trains the round ends admitted more disturbance. It is felt that the explanation is that the round ends serve as funnels to admit a longer section of wave crest and that instead of being 600 ft. gate openings, they are effectively nearly 1600 ft. gate openings. The use of round ends on the wide mole, therefore, appears definitely disadvantageous.

2. RESULTS OF STUDIES OF MODEL 3

(a) Physical differences between Model 2 and Model 3. In section VII-A, it was pointed out that both Model 2 and Model 3 are essentially the same and that the only difference between them is in the depth of dredging within the mole area. This statement does not refer to any of the details of the mole construction, since these will be described specifically as they are tested. The difference in the basic configuration of the model is shown in Figure 58, Page 85. The test conditions remain the same as they were for Model 2, i.e., wave trains having 600 ft. wave length and periods of about fifteen seconds, surge wave trains of three minute periods, and surge wave trains of six minute periods. The six minute surges were used because, as was pointed out under paragraph (b) of the results of Model 2, the six minute period produces the most severe conditions that can be encountered in the basin. The amplitudes of these wave and surge trains were kept at the same values that were used for Model 2. It was decided to make most of the tests using only one wave period at a time instead of combining, for example, the fifteen second waves and the three minute surges, because it was felt desirable to separate and evaluate the amount of disturbance resulting from each. This was considered to be consistent with the physical situation known to exist in the harbor, i.e., that there appears to be little correlation between the appearance of the maximum amplitudes of the fifteen second waves and the maximum amplitudes of the other period surges. Furthermore, tests in the basin also showed that the disturbances from the different trains were additive, at least within the accuracy of the model studies. Therefore, it was concluded that more information could be obtained by studying each source of disturbance separately.

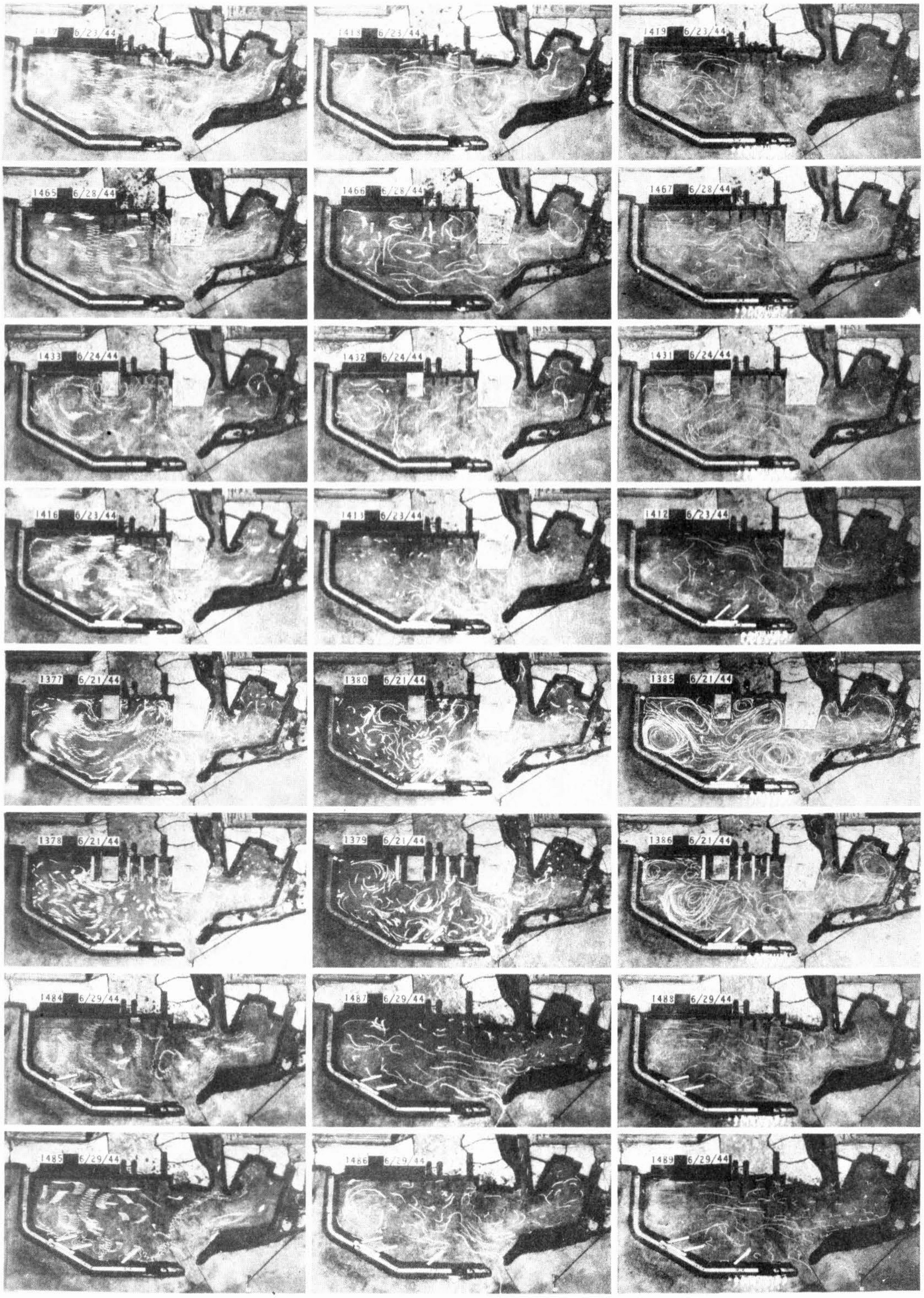
(b) Effect of deepening of the basin. Check runs were taken to ascertain the difference in the disturbance pattern which resulted from the dredging. Figures 100 to 102, inclusive, show the disturbance without the mole in place for the fifteen second, the three minute and the six minute trains. In each case the dotted lines give the contours of constant vertical amplitude for the 35 ft. depth of Model 2 and the solid line shows the same information for Model 3 with portions of the area dredged to 45 ft. It will be noted in all cases that the disturbance pattern is essentially the same, although there appears to be a slight tendency for the magnitudes to be somewhat lower for Model 3.

Similar conditions were found to exist with the mole in place. Figures 103 and 104 show the vertical amplitudes for the fifteen second and the three minute trains with a 2070 ft. gate. It will again be noted that the deeper basin shows slightly lower amplitude, especially at points of maximum disturbance such as can be seen directly inside the mole gate for the fifteen second waves.



15 SECOND WAVE | 3 MINUTE SURGE | 6 MINUTE SURGE

FIG. 96 HORIZONTAL MOVEMENT WITH THE 2070 FT. GATE OPENING



6 MINUTE SURGE | 3 MINUTE SURGE | 15 SECOND WAVE

FIG. 97 HORIZONTAL MOVEMENT WITH THE 600 FT. GATE OPENING

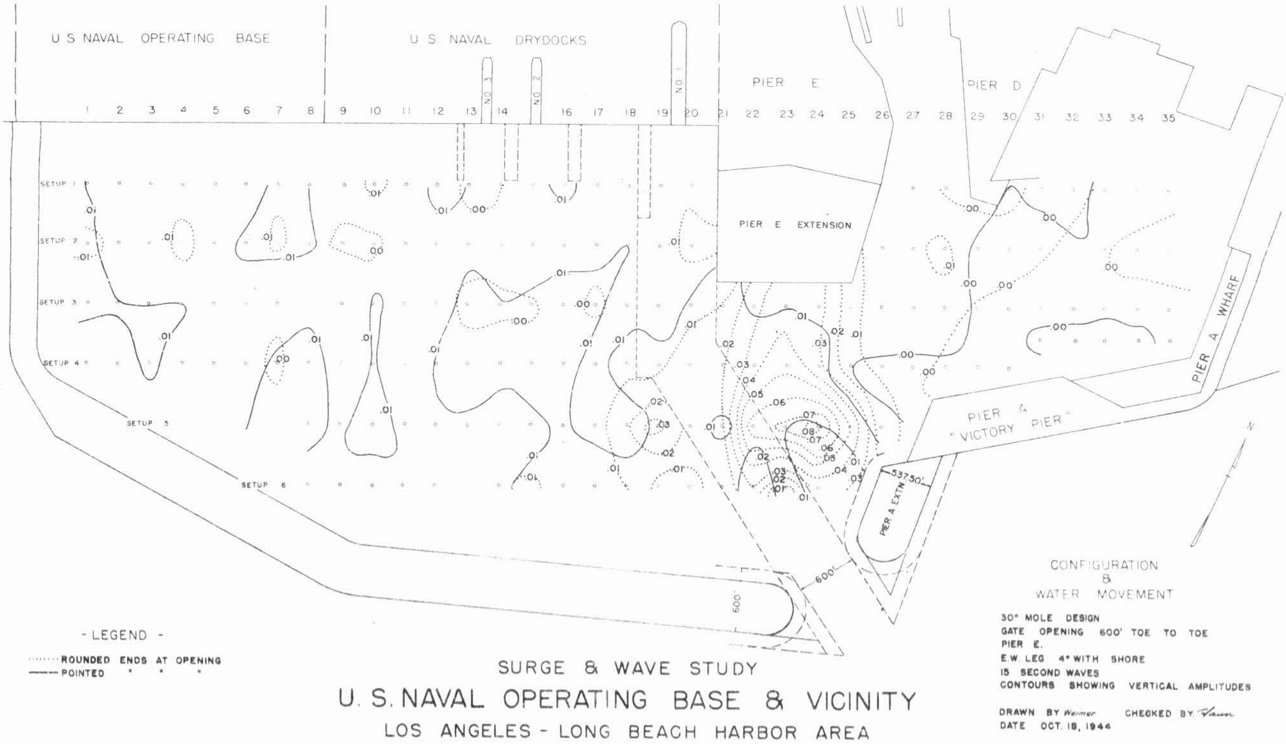


FIG. 98 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
SEMICIRCULAR VS. POINTED MOLE ENDS

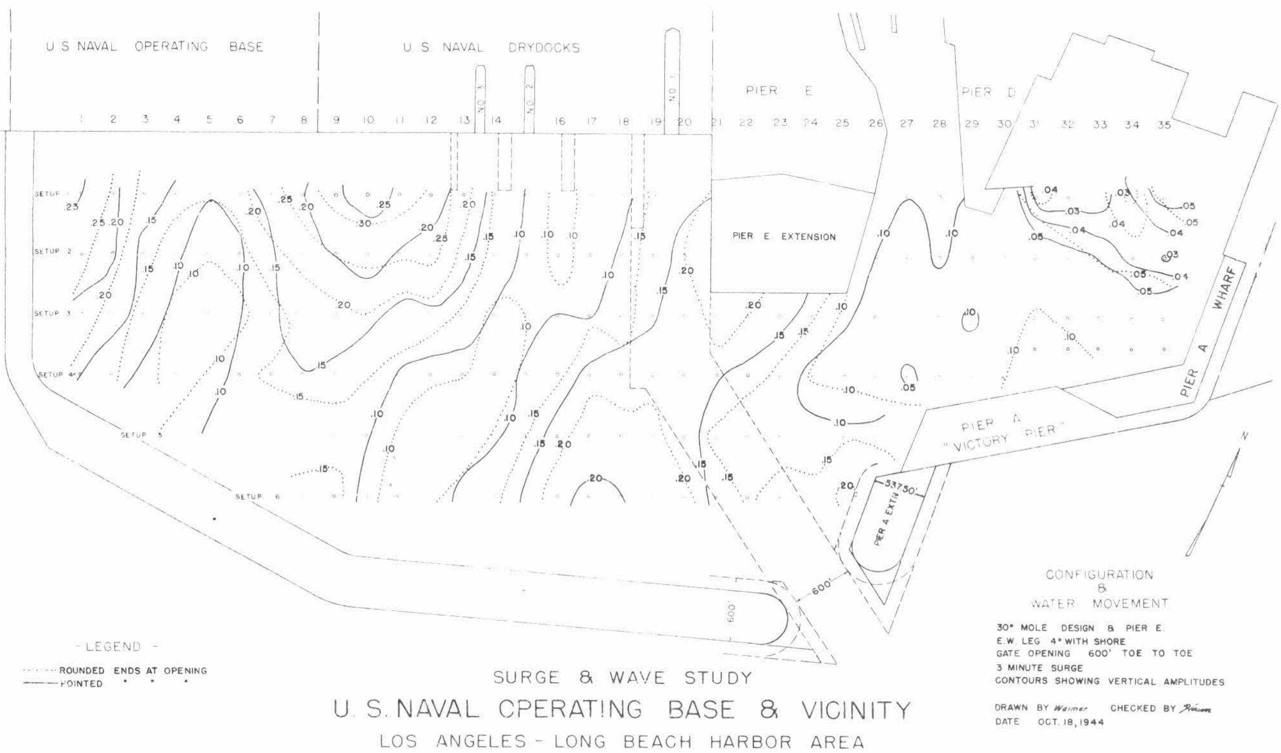


FIG. 99 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
SEMICIRCULAR VS. POINTED MOLE ENDS

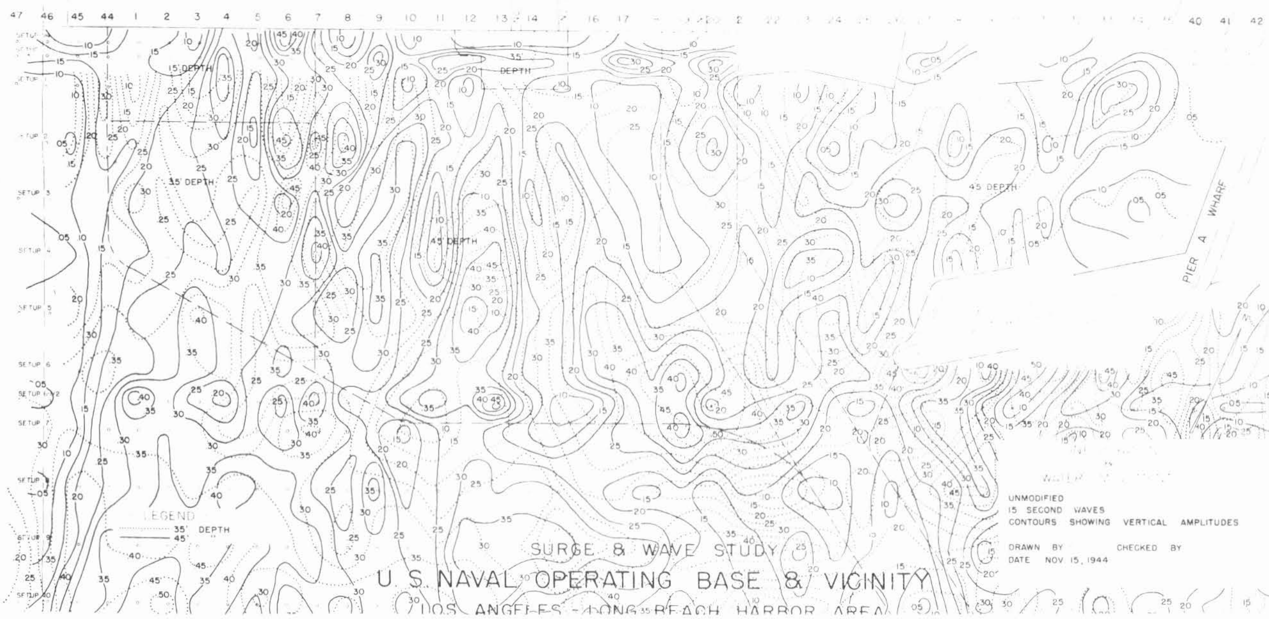


FIG. 100 VERTICAL MOVEMENTS CAUSED BY 15 SECOND WAVES
UNMODIFIED BASIN 35 FT. DEPTH VS. 45 FT. DEPTH

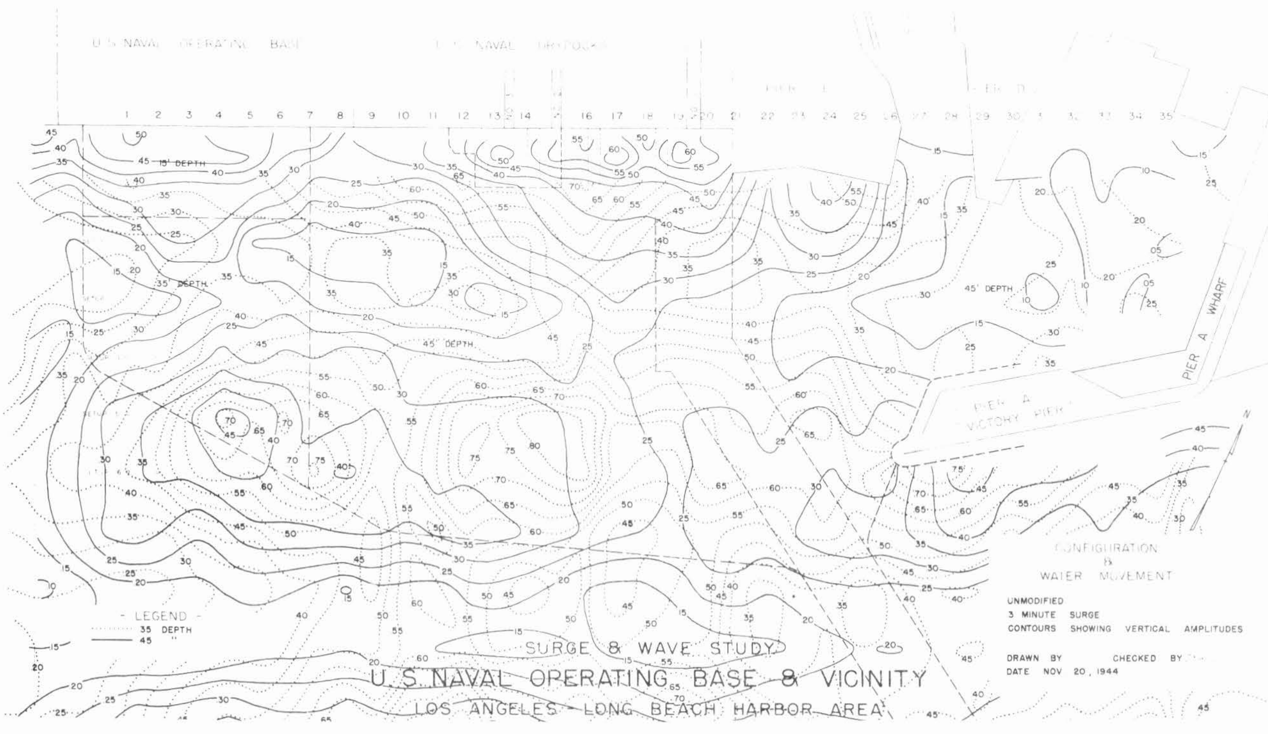


FIG. 101 VERTICAL MOVEMENTS CAUSED BY 3 MINUTE SURGE
UNMODIFIED BASIN 35 FT. DEPTH VS. 45 FT. DEPTH

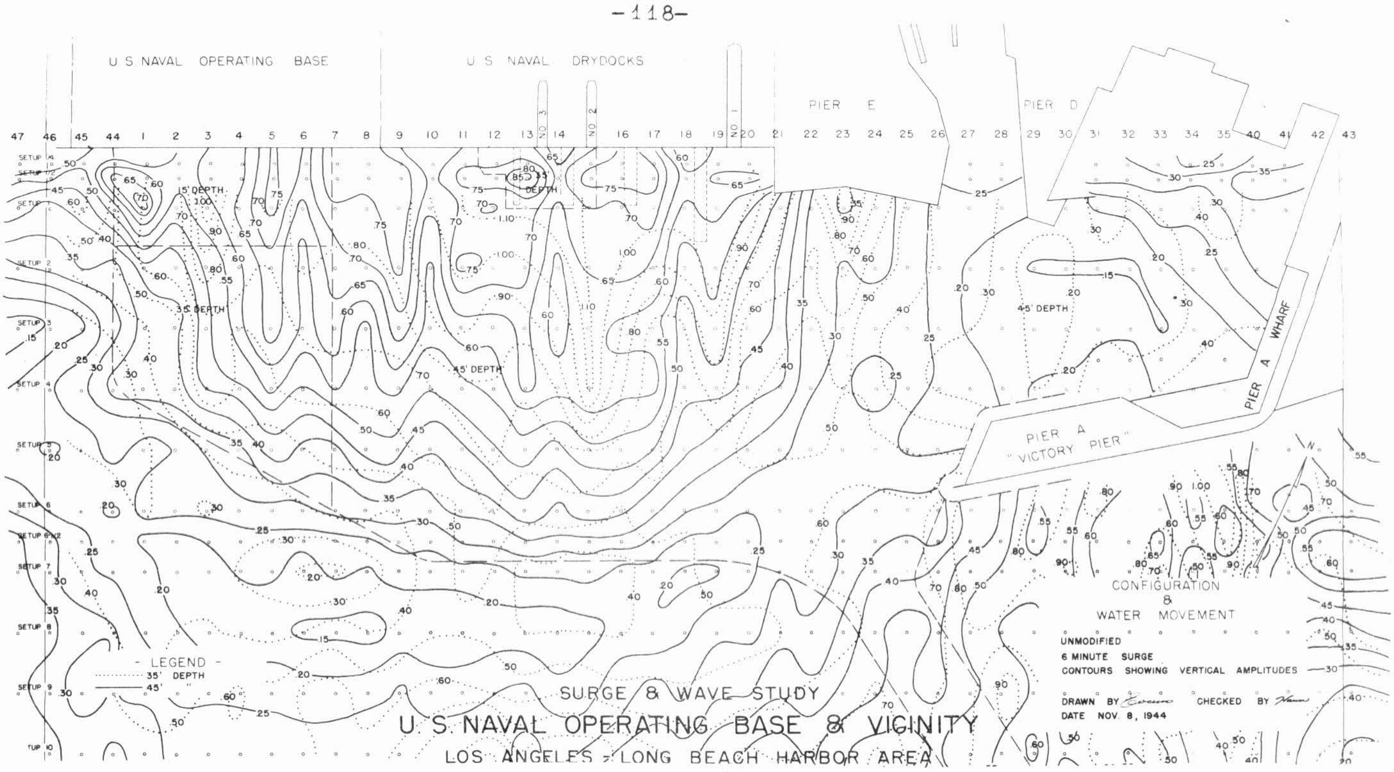


FIG. 102 VERTICAL MOVEMENTS CAUSED BY 6 MINUTE SURGE UNMODIFIED BASIN 35 FT. DEPTH VS. 45 FT. DEPTH

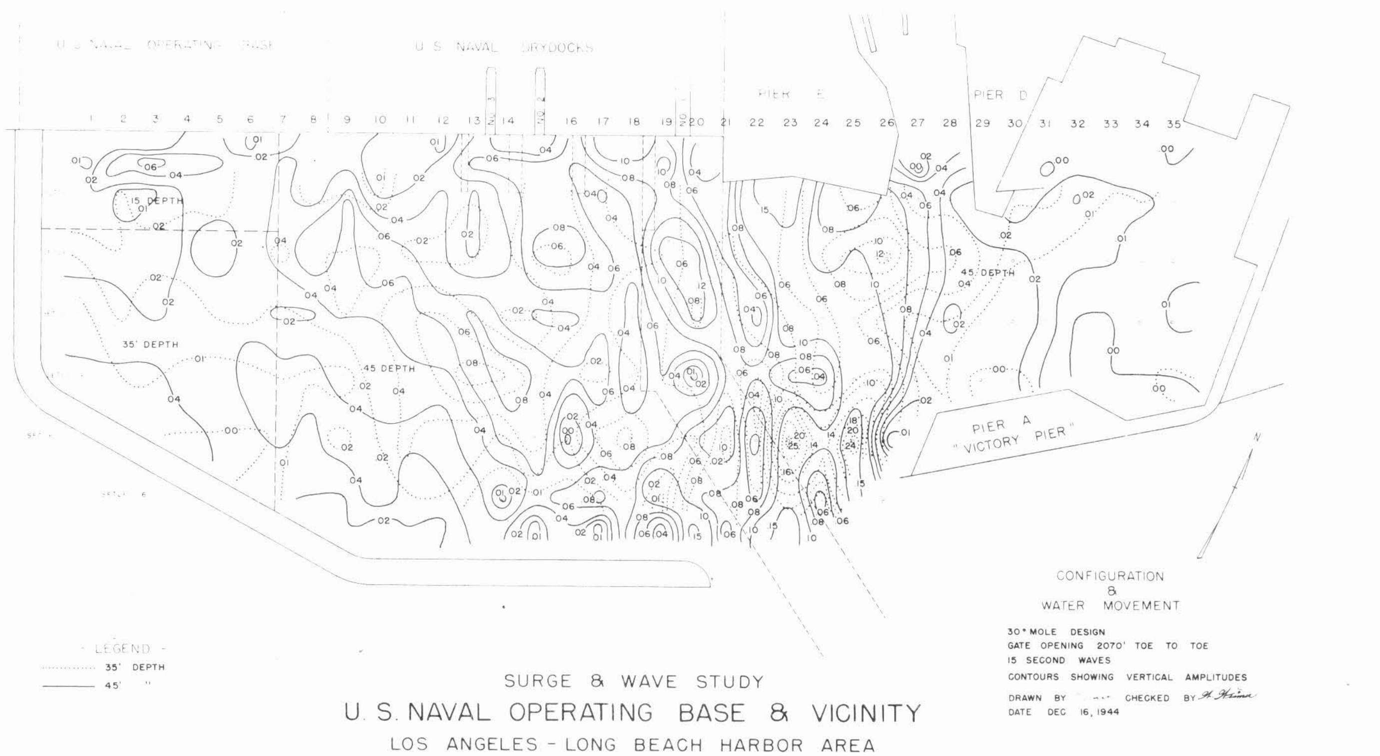


FIG. 103 VERTICAL MOVEMENTS CAUSED BY 15 SECOND WAVES MODEL NO. 2, 35 FT. DEPTH VS. MODEL NO. 3, 45 FT. DEPTH

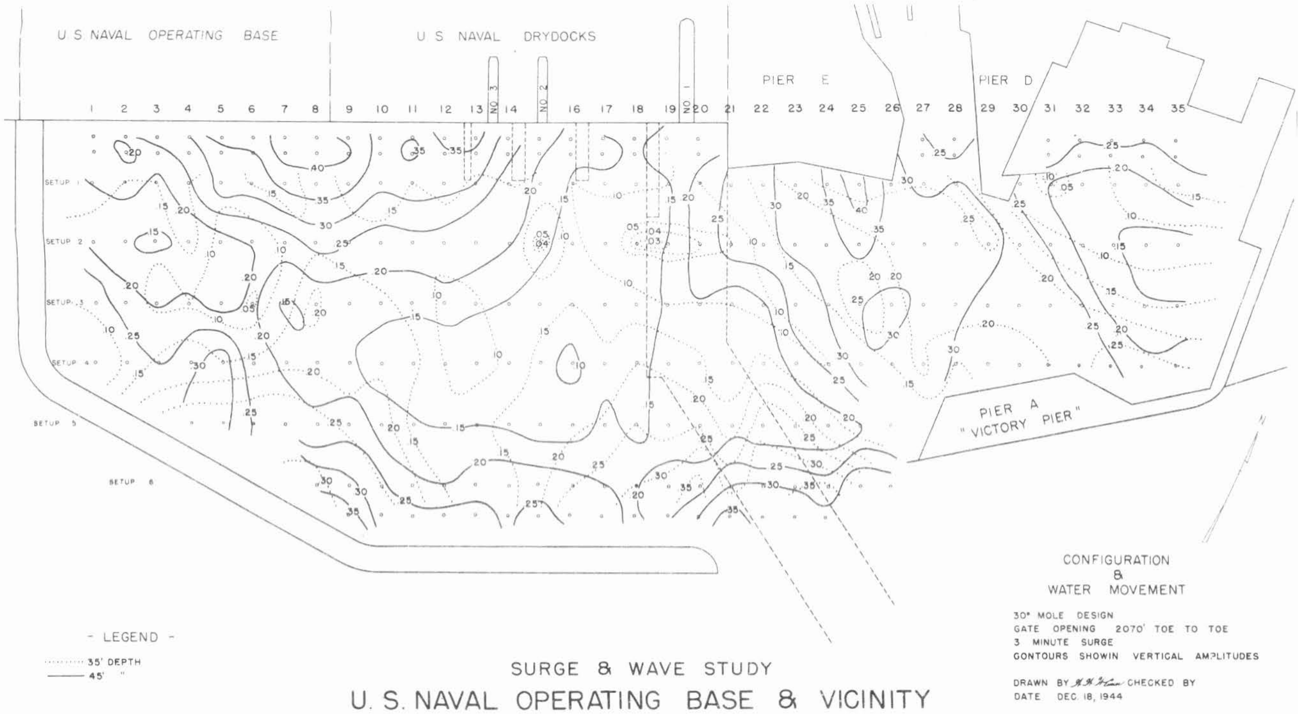


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FIG. 104 VERTICAL MOVEMENTS CAUSED BY 3 MINUTE SURGE MODEL NO. 2, 35 FT. DEPTH VS. MODEL NO. 3, 45 FT. DEPTH

The comparison in the horizontal motion is given by Figures 105 and 106, which show the reaction of the two basins to the three minute surge. It will be seen that the general effect of this deepening of the major portion of the basin within the mole is to improve conditions within the area by decreasing the amount of movement produced for a given energy input from a wave or a surge train.

(c) Effect of mole widening. At the same time that the designers at the Naval Operating Base proposed to deepen the channel within the mole, the suggestion was made also that the mole be widened and that this widening should be accomplished by changing the seaward side of the mole, thus leaving the basin dimensions unchanged. The effect of this proposal on the behaviour of the basin was investigated in the model. However, no study was made to see if a difference could be detected in the disturbance pattern on the seaward side.

Since this widening did not affect the dimensions of the basin itself, it is obvious that the differences in basin behaviour, if they exist, must be traceable to changes in conditions at the gate. These gate conditions will be modified by the widening of the mole because the widening changes both the shape and the location of the opening. The width of the mole as originally designed was to be 287.5 ft. The proposed broadening of the mole increased this to 537.5 ft. It will be seen that this wide mole is, therefore, approximately as wide as the wave length

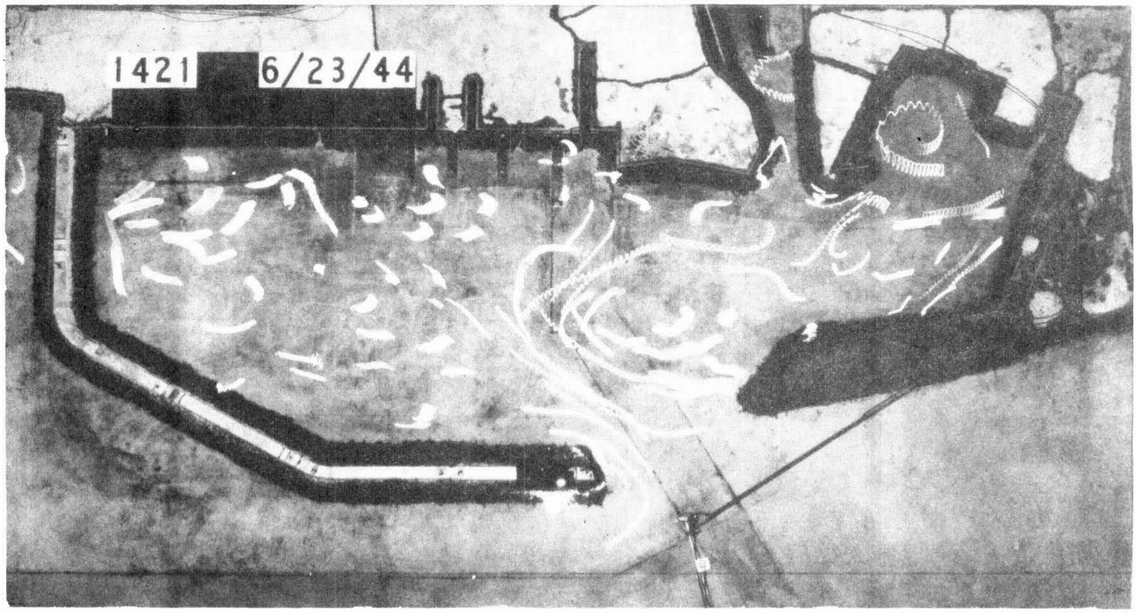


FIG. 105 HORIZONTAL MOTION CAUSED BY 3 MINUTE SURGE
MODEL NO. 2, 35 FT. DEPTH

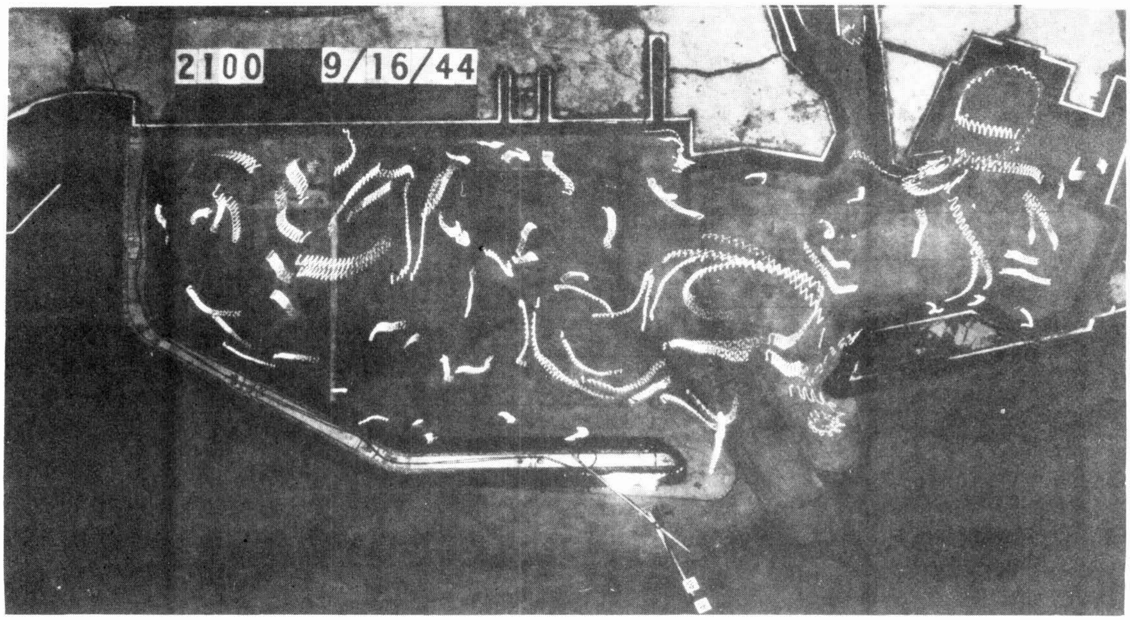


FIG. 106 HORIZONTAL MOTION CAUSED BY 3 MINUTE SURGE
MODEL NO. 3, 45 FT. DEPTH

of the fifteen second train. With a mole of this width it is obviously possible to design the mole ends so as to give several configurations that have significant geometric differences. The behaviour of the basin was compared for a series of these configurations. The clearest results are obtained from the tests using the three minute surges.

(1) Relative effectiveness of semicircular vs. pointed mole ends. Figure 107 shows the comparison of the effect of the semicircular mole ends with the moles in which the ends were cut off so that they were parallel with the 45 ft. channel to the east gate. It will be seen that the latter configuration results in the least disturbance within the basin. However, this latter construction offered some disadvantages from the point of view of navigation, since the basin entrance, in effect, is a short, narrow channel which would require more care and skill to negotiate than an opening having no appreciable length in the direction of the ship's travel.

(2) Semi-elliptical mole ends. An attempt was made to combine the advantages of both openings by employing a semi-elliptical end for the mole. This is shown in Figure 108 which compares the behaviour of this design with that of the round end. The comparison is made for the 1320 ft. gate opening instead of the 750 ft. width, because comparable data were not available for the smaller opening. The wider opening would tend to minimize the difference in behaviour due to the shape of the mole ends. This increases the significance of the fact that the movements in the basin are slightly, but significantly, lower with the elliptical ends.

(3) Quantitative evaluation of effectiveness of semi-circular and semi-elliptical mole ends. An attempt was made to obtain a more quantitative comparison of these two mole end designs. As will be seen from an examination of any of the contour maps of the vertical motion, the measuring elements, shown by the small circles, were located on a series of lines parallel to the shoreline of the Naval Operating Base and the drydocks. These lines are designated as set-ups 1 to 6. Averages of the motions shown along each individual set-up line were taken for the two designs of mole ends. This was done both on Model 2 and Model 3. On Model 2 these computations were made both for the fifteen second waves and the three minute surges. For the three minute surge each set-up average showed a lower value for the pointed ends than for the rounded. The average for the entire basin of the vertical motion for the pointed ends was 85% of that with the rounded. For the measurement with the fifteen second wave train the indications were not so clean-cut. However, the average of the basin showed nearly the same thing, i.e., that the motion for the pointed ends was about 89% of that for the round ends. On Model 3 measurements were made only for the three minute surge. On this study there was little discernible difference on the over all average for the two ends. However, one striking fact was noted. Along the line of set-up #1 the pointed ends showed the motion reduced to 85% of that with the rounded ones. On set-up

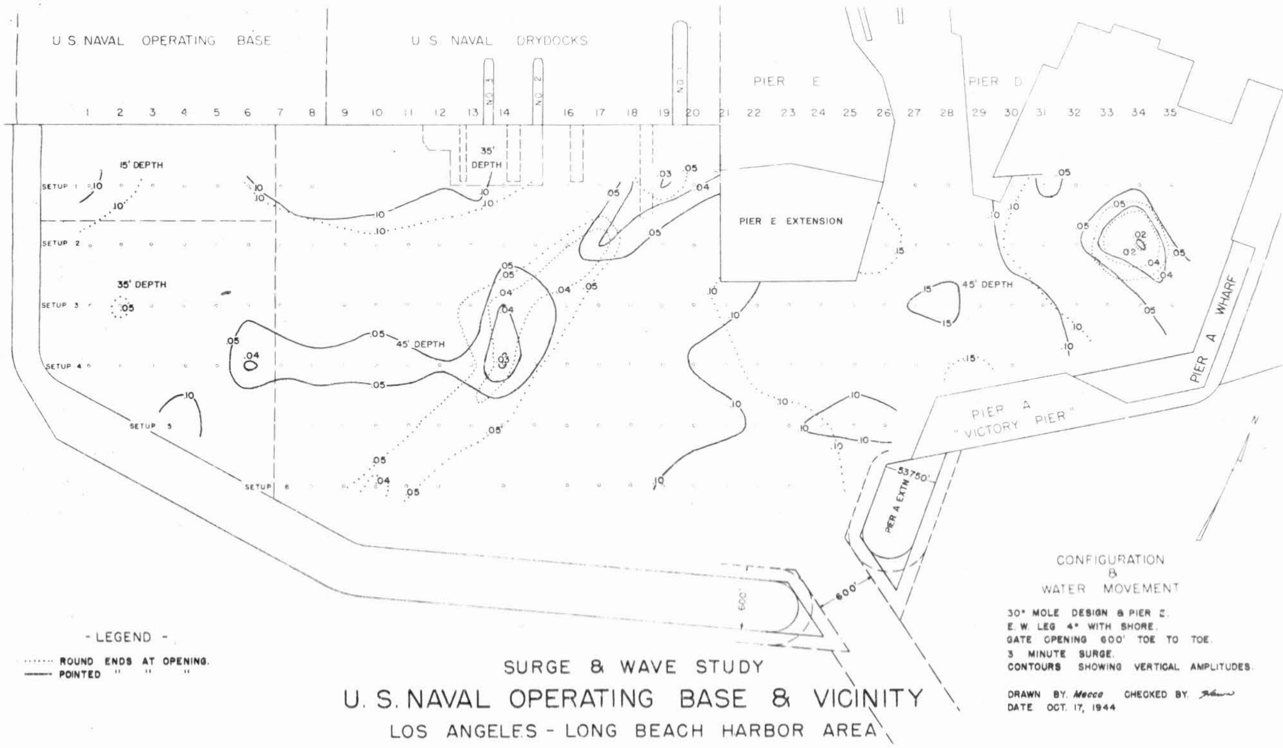


FIG. 107 VERTICAL MOVEMENTS CAUSED BY 3 MINUTE SURGE
SEMICIRCULAR VS. POINTED MOLE ENDS

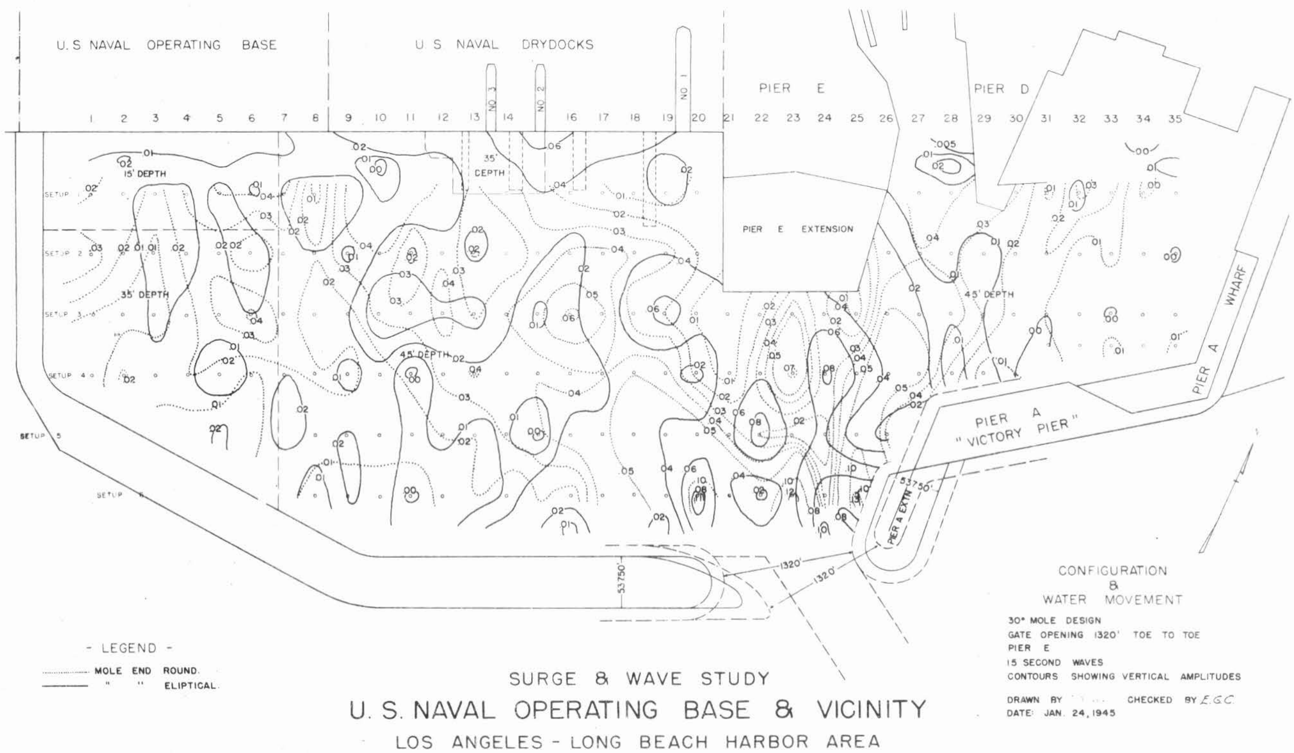


FIG. 108 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
ROUND MOLE ENDS VS. SEMIELLIPTICAL MOLE ENDS

#3 the motions were nearly equal. On set-up #5 the motion for the pointed ends was about 104% of that with the rounded ends, whereas on setup #6 the motion for the pointed ends was about 140% of that with the rounded ends. In other words, for conditions along the shoreline of the drydocks and the Naval Operating Base areas, the pointed ends produce a significant improvement, whereas this effect decreased towards the gate and close to the gate the motion was greater for the pointed ends. It is felt that the conditions in the drydock and Naval Operating Base areas are so much more critical than those near the gate that the good effects of the pointed ends in this area far outweigh any opposite effect they may have near the gate. The improvement in conditions caused by using the pointed ends is also in accordance with the known behaviour of the wave trains.

(d) Width of gate opening. In the course of the model study it became increasingly clear that the width of the mole gate opening was one of the most important of the factors controlling the condition of the basin within the mole. Visual observations pointed to the necessity of having this gate as small as practicable. This conclusion was confirmed by a study of the contour maps of the vertical motion for the different gate openings that were available. In order to get a more complete evaluation of the relationship between the width of the gate opening and the water movements within the basin, a special study was undertaken with the gate opening as the only variable. For this study the wide mole design was employed with semielliptical ends in accordance with the findings of the previous paragraphs and with no internal construction in the basin except Pier A wharf and Pier E extension. It was decided that these should be incorporated since by this time they were authorized and under construction and, therefore, would form a permanent part of the basin boundaries. The gate openings investigated in this study were 100 ft., 200 ft., 300 ft., 400 ft., 500 ft., 600 ft., 750 ft., 1320 ft., and 2070 ft. from toe to toe of the slope. Figures 109 to 117, inclusive, show photographs of the various gate openings with fifteen second wave trains coming from both gates.

(1) Comparison of vertical water movements in critical basin areas. In order to study in detail the effect of the change in gate opening, a series of critical areas was selected in the basin. For each area the average vertical movement was computed from the measurements made with the wave elements located in the area. The areas selected were as follows:

(a) Drydock area
(b) Naval Operating Base corner
(c) North-south leg of the mole
(d) 30° inclined leg of the mole
(e) East-west leg of the mole
(f) Long Beach harbor area
(g) The south front of Pier E extension

In addition, the average of the elements in the entire basin was computed to give an over-all figure of merit. Figure 118 shows

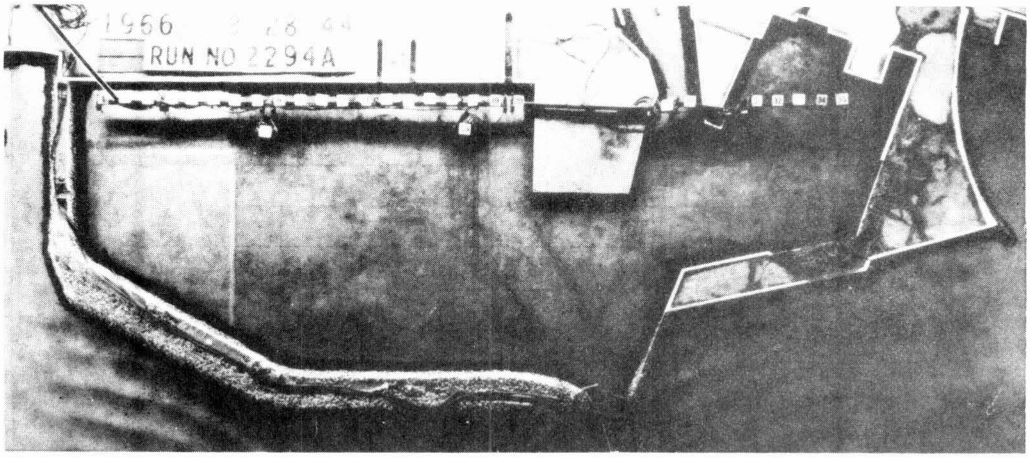


FIG. 109 100 FT. GATE OPENING

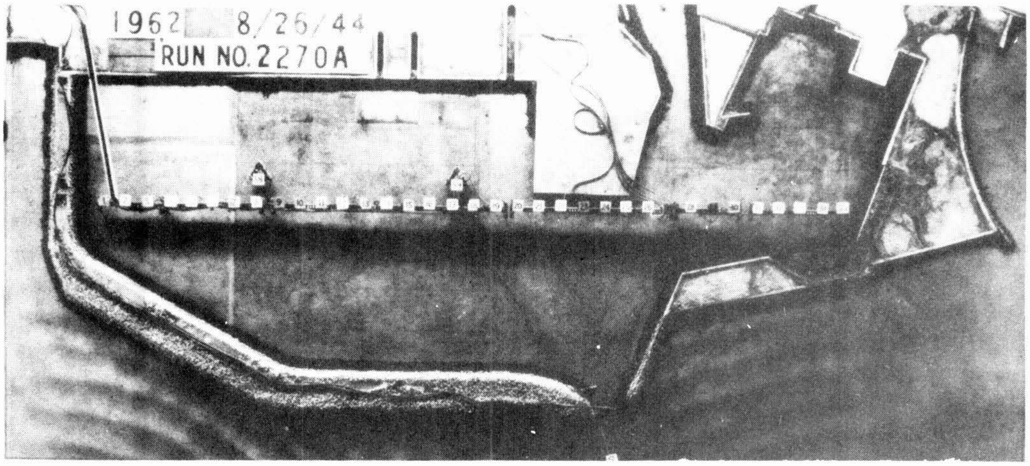


FIG. 110 200 FT. GATE OPENING

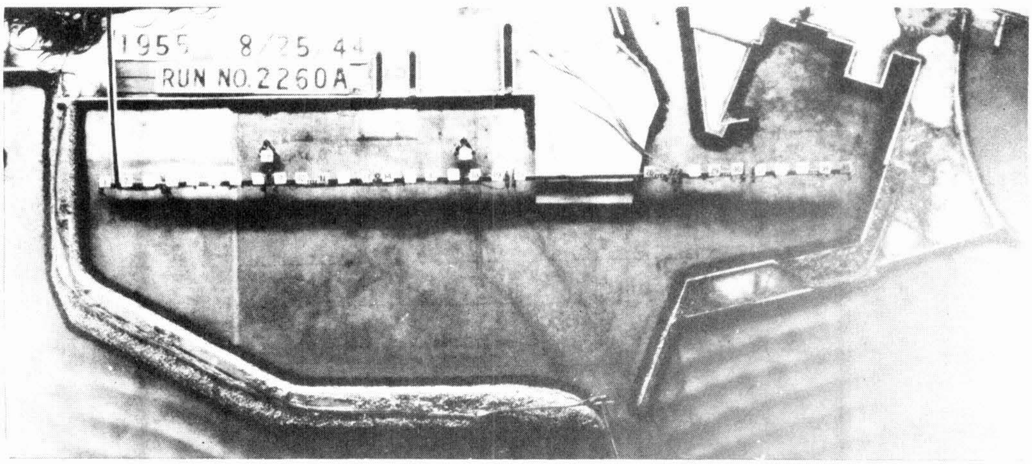


FIG. 111 300 FT. GATE OPENING

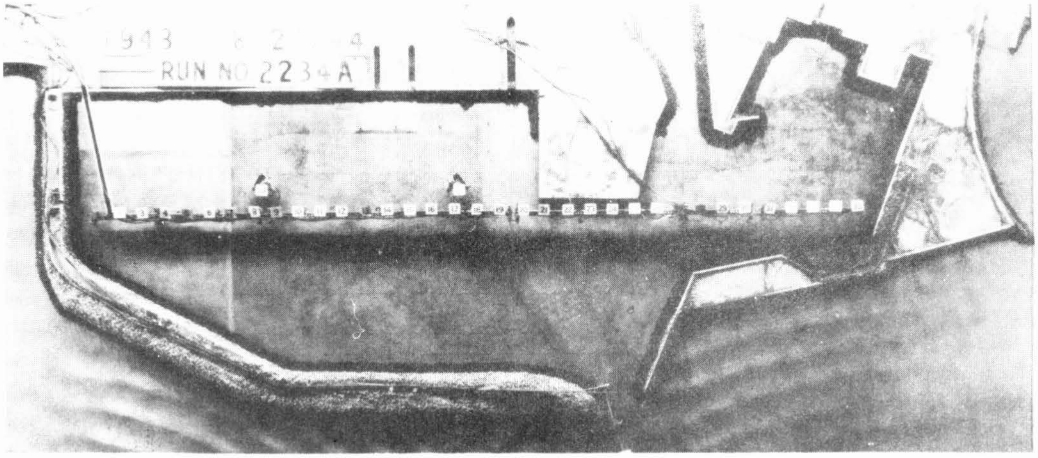


FIG. 112 400 FT. GATE OPENING

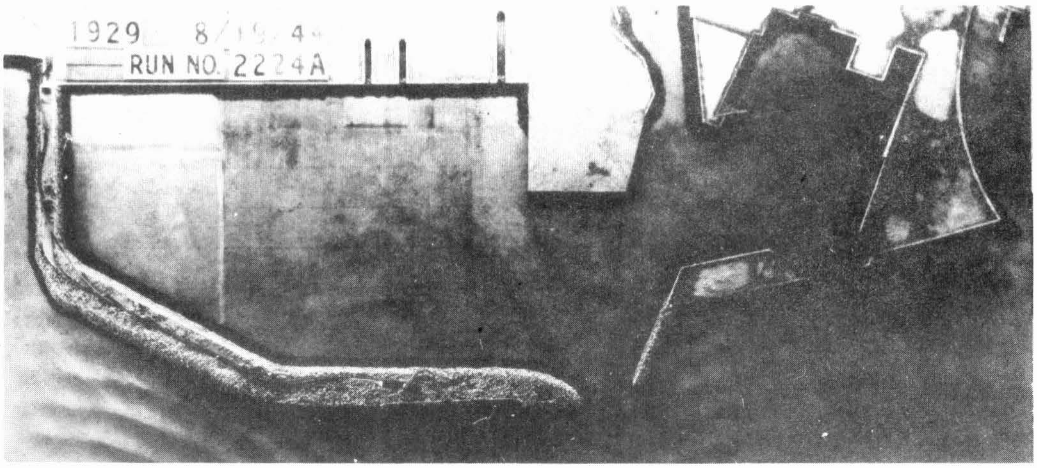


FIG. 113 500 FT. GATE OPENING

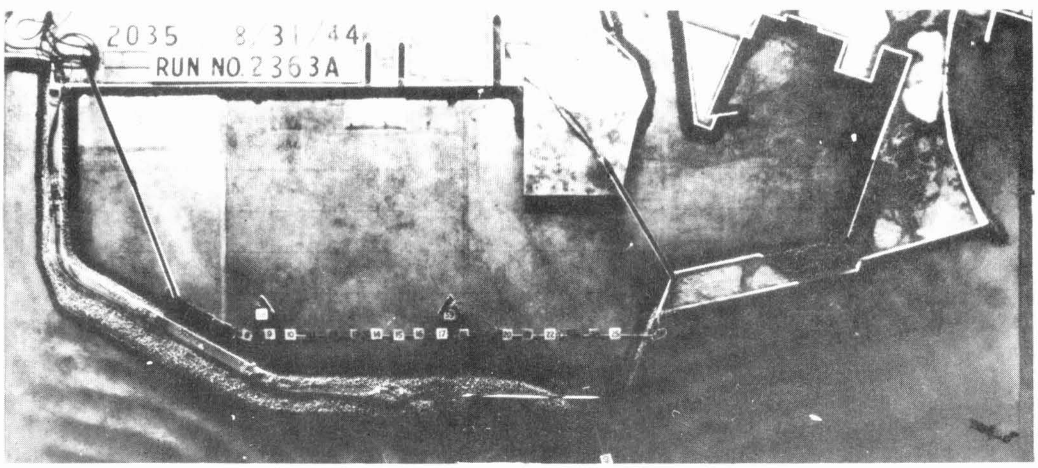


FIG. 114 600 FT. GATE OPENING

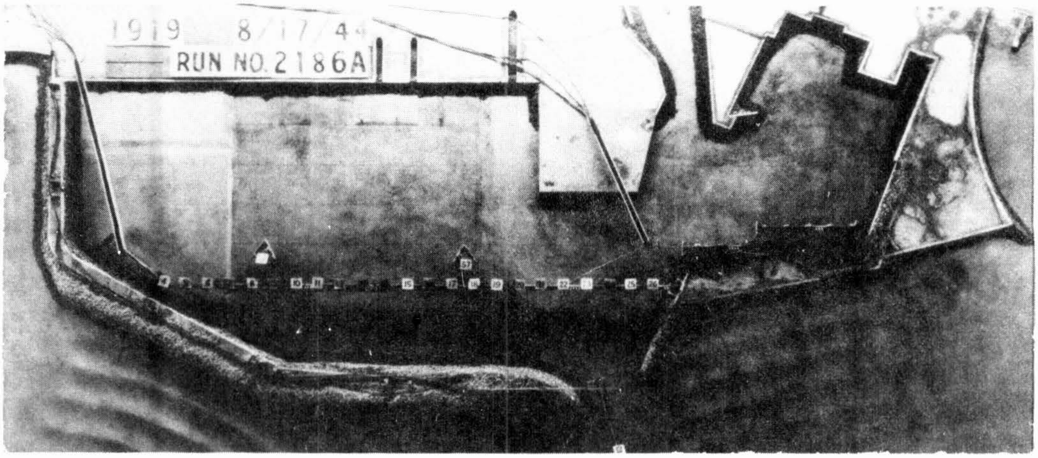


FIG. 115 750 FT. GATE OPENING

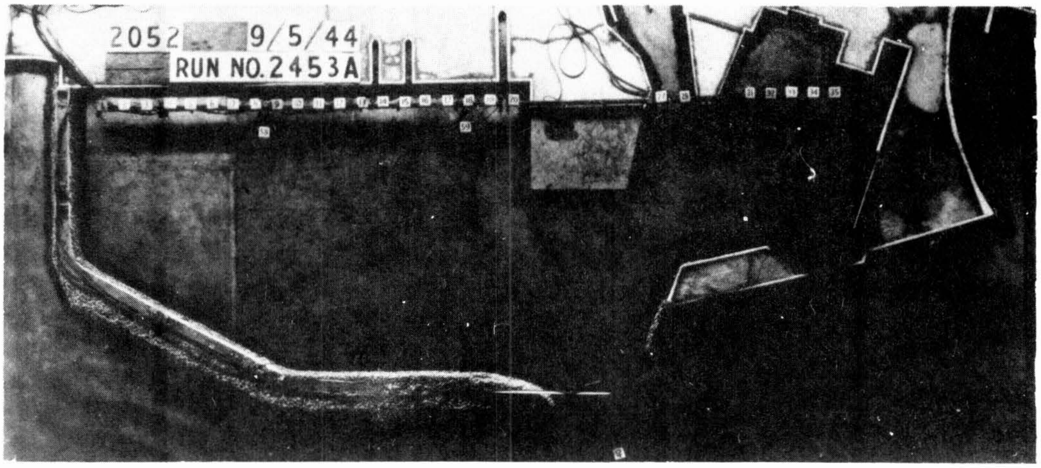


FIG. 116 1320 FT. GATE OPENING

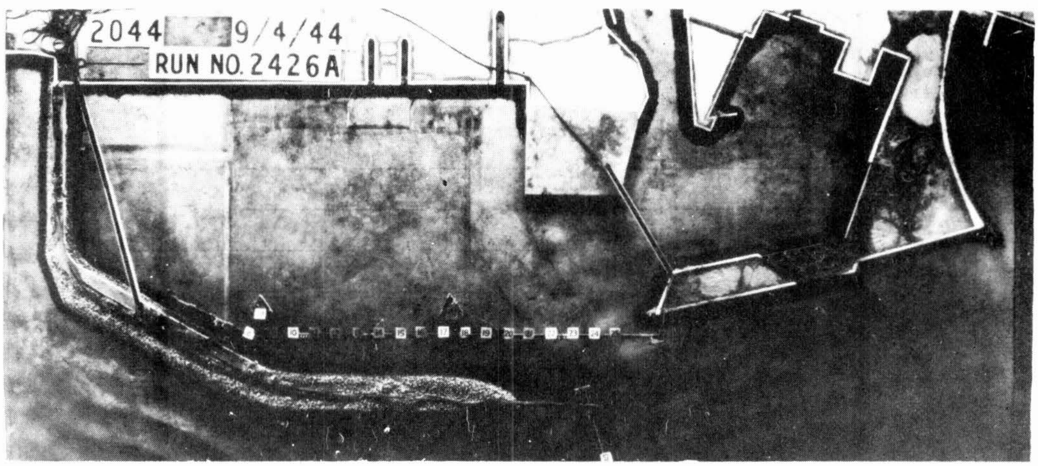


FIG. 117 2070 FT. GATE OPENING

the boundaries of these areas and the measuring elements used in obtaining the averages. The results are shown on Figures 119, 120, and 121 for the fifteen second wave trains, the three minute surge trains, and the six minute surge trains, respectively. It will be observed that the ordinate gives the wave height in the different areas, plotted in percent. The movement in the drydock area with a 750 ft. gate opening was taken as the standard of comparison; the amplitude being defined as 100%. This area was used because it had been defined by the Naval Operating Base as the most critical area in the basin. Furthermore, the 750 ft. gate opening was chosen as the standard because the Laboratory had been informed that this represented the minimum opening that could be used without sacrifice of ease of navigation. The comparison of these three charts is very enlightening.

<u>(2) Relation between period of waves and effectiveness of gate opening.</u> In the first place, it will be observed that the effect of the width of the gate opening varies very greatly with the period of the wave train. The disturbance due to the fifteen second waves is greatly decreased even with the widest gate opening. On the other hand, for the three minute surges it will be seen that the 2070 ft. opening reduces the motion only to about 50% of what it is with no mole at all, whereas the 750 ft. gate opening brings the disturbance down to about 20% of its original value. For widths less than 750 ft., the improvement of conditions in the basin is quite slow. In fact, it is necessary to reduce the width to 200 ft. to decrease the vertical motion to about one-half the value it has at 750 ft.

<u>(3) Reaction of basin to six minute surges.</u> Figure 121 serves to emphasize the seriousness of the situation which can be expected to exist in case a series of the six minute surges occurs in the harbor.

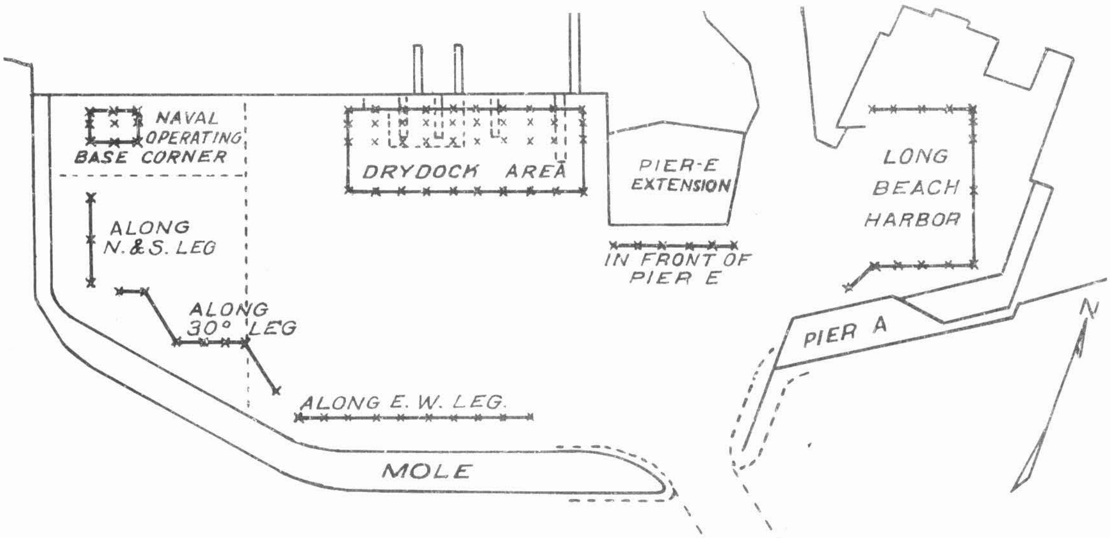


FIG. 118 ELEMENT GROUPS USED FOR OBTAINING AVERAGE VERTICAL AMPLITUDES IN VARIOUS BASIN AREAS

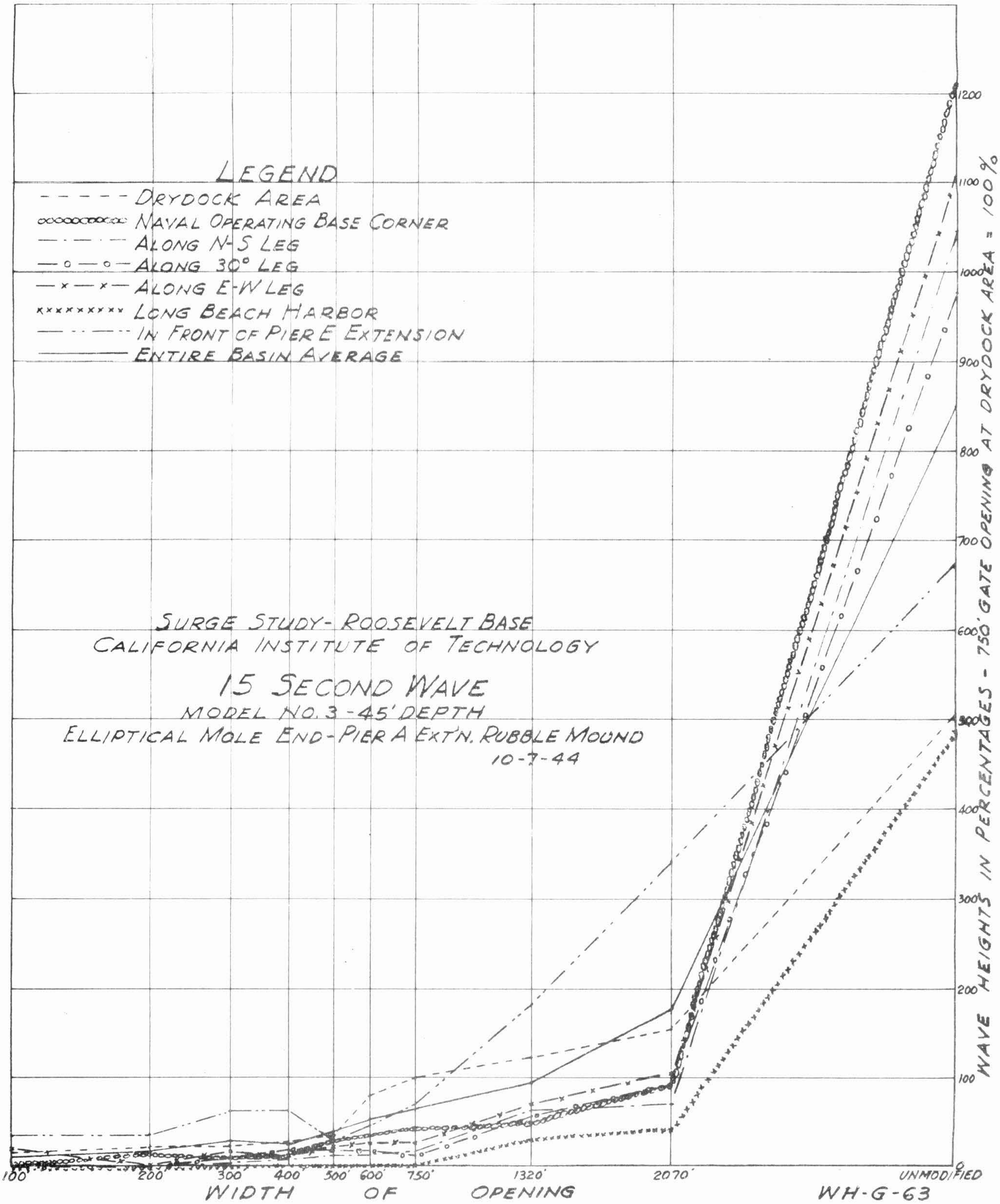


FIG. 119 COMPARISON OF VERTICAL AMPLITUDES
VARIOUS BASIN AREAS - 15 SECOND WAVES

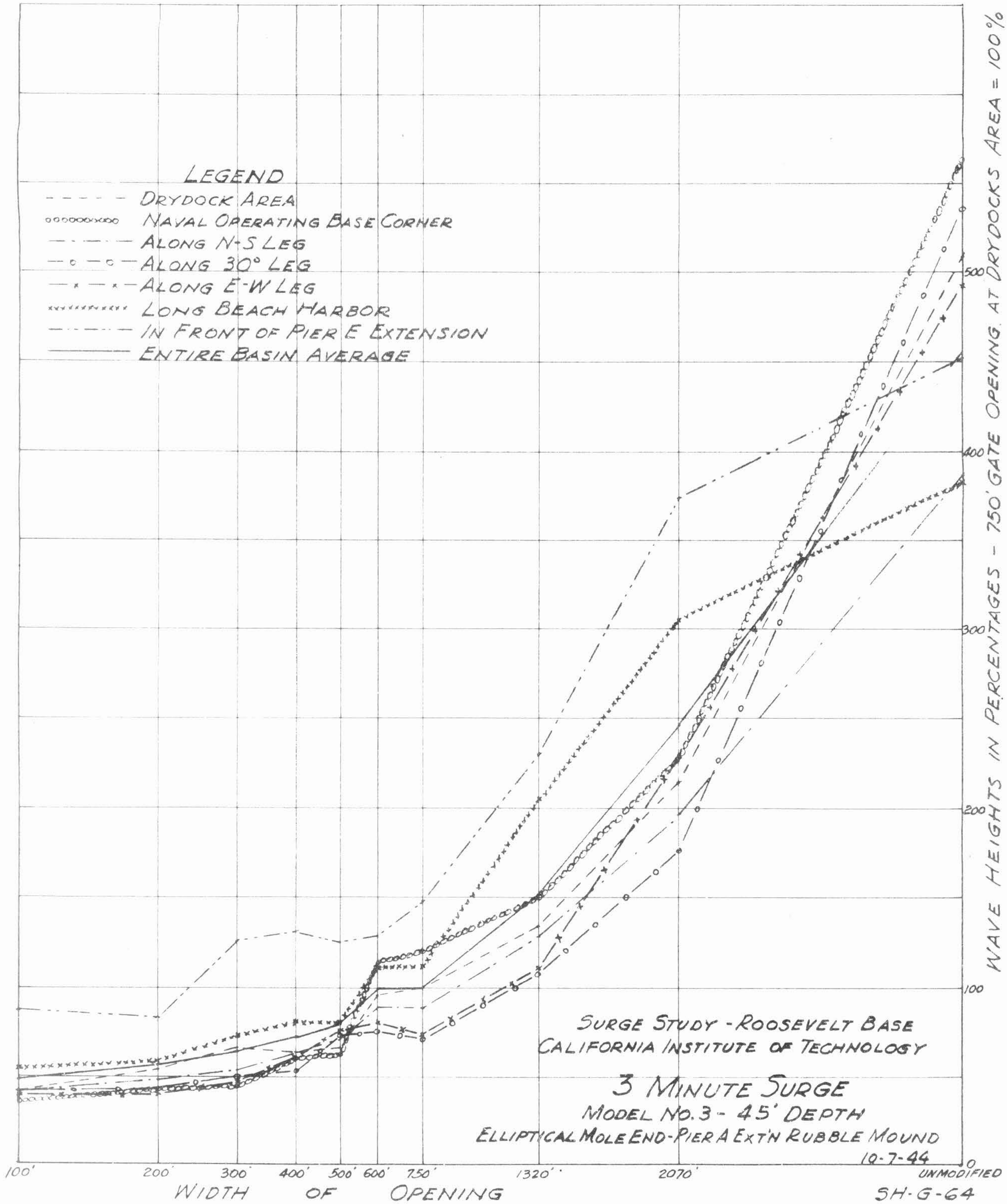


FIG. 120 COMPARISON OF VERTICAL AMPLITUDES
VARIOUS BASIN AREAS - 3 MINUTE SURGE

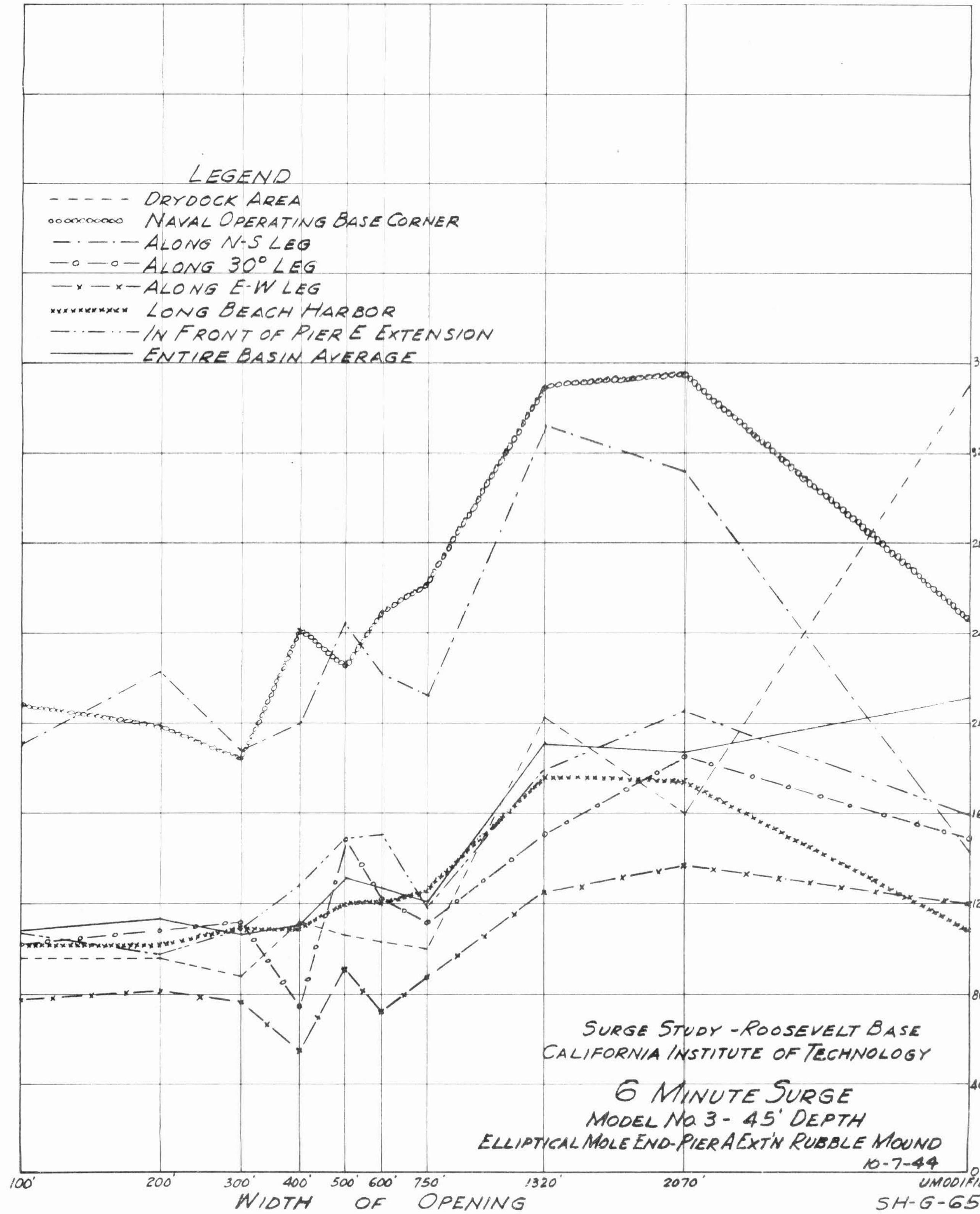


FIG. 121 COMPARISON OF VERTICAL AMPLITUDES
VARIOUS BASIN AREAS - 6 MINUTE SURGE

From this figure it will be seen that conditions within the mole are, on the average, only slightly improved by the presence of the mole over what they would be without the mole in place, even for the smallest gate opening. The only significant exception to this statement occurs in the drydock area itself. At this point it will be seen that the disturbance with the 750 ft. gate or any smaller opening is less than a third of what it would be with no mole. However, this statement is slightly misleading, because a further inspection will show that the drydocks area is apparently a location of especially high vertical movement when the mole is not in place. With the mole installed and the 750 ft. gate opening, this high value is reduced to about the average for the other areas in the basin. It is also interesting to note that with the mole in place there are two areas of outstandingly high amplitude, i.e., the northwest corner of the basin which is designated as the Naval Operating Base corner, and the area along the north-south leg of the mole. It will be seen that for both the 1320 ft. and the 2070 ft. openings the disturbance in these two locations is considerably greater than it is without the mole in place and that even for the smallest gate openings investigated, the motion is still nearly as large as it is with no mole.

(4) Optimum gate opening. From this study it can be seen that all of the factors point in one direction, i.e., to the choice of a relatively narrow gate opening. Considering all the possibilities of disturbance, it appears that the maximum width of opening that should be considered is 750 ft. On the other hand, the improvements secured by reducing the opening below the 500 ft. width are probably too small to justify the accompanying handicap to navigation.

(5) Relative motion in various areas in the basin. Figures 119 to 121, inclusive, also serve to give a comparison of the relative quietness of the different areas of the basin. This information should be taken into consideration in planning for the future use of the basin. For example, it will be noted that both for the fifteen second waves and the three minute surges, the areas along the 30° leg and the east west leg of the mole are particularly quiet.

(6) Details of response of basin for various gate openings. Comparison maps showing contours of the motion in the entire basin have been prepared for six different gate openings. In each case, the behaviour of the given gate opening is compared to that for the 750 ft. gate opening. Figures 122 to 124, inclusive, are for the 200 ft. opening, Figures 125 to 127, inclusive, the 400 ft. opening, Figures 128 to 130, inclusive, the 600 ft. opening, Figures 131 to 133, inclusive, the 1320 ft. opening, and Figures 134 to 136, inclusive, the 2070 ft. opening.

(e) Effect of additional structures within the basin

(1) Pier A wharf. Paragraph (e) of the discussion of the results of studies of Model 2 presents an evaluation of the effect of the

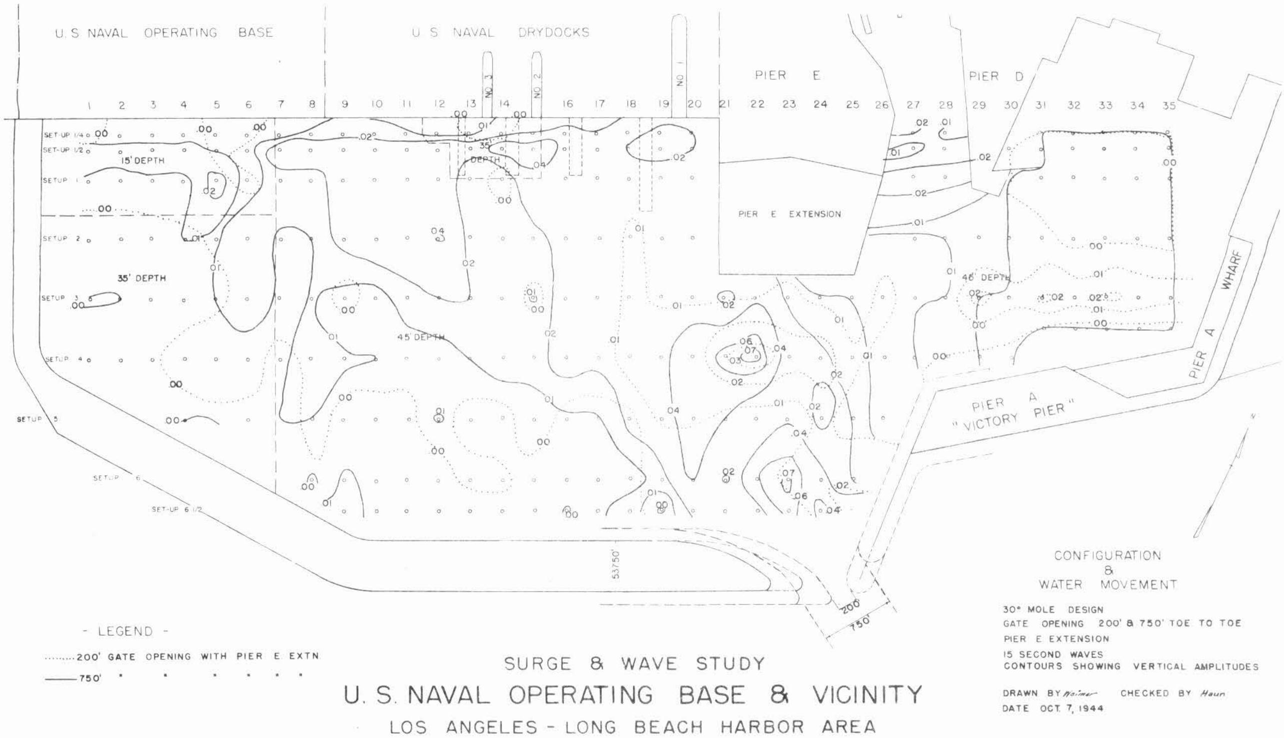


FIG. 122 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
200 FT. GATE OPENING VS. 750 FT. GATE OPENING

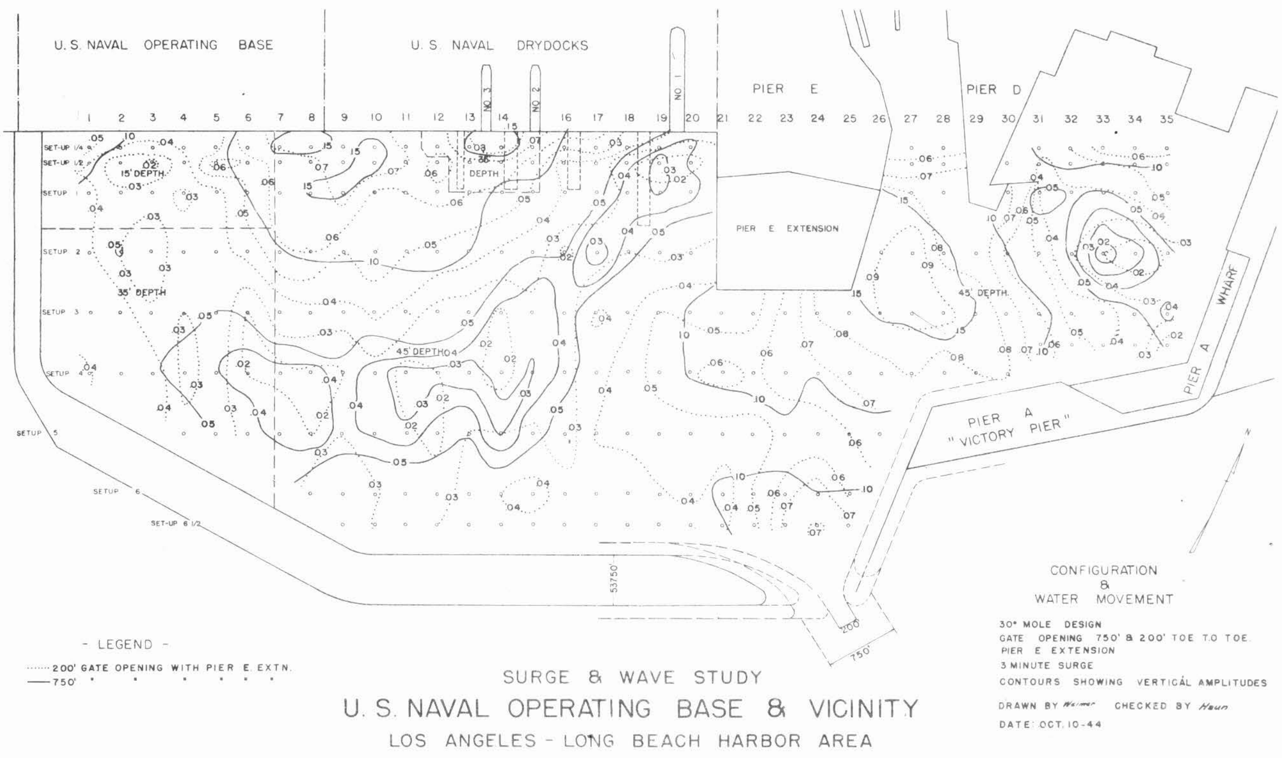


FIG. 123 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
200 FT. GATE OPENING VS. 750 FT. GATE OPENING

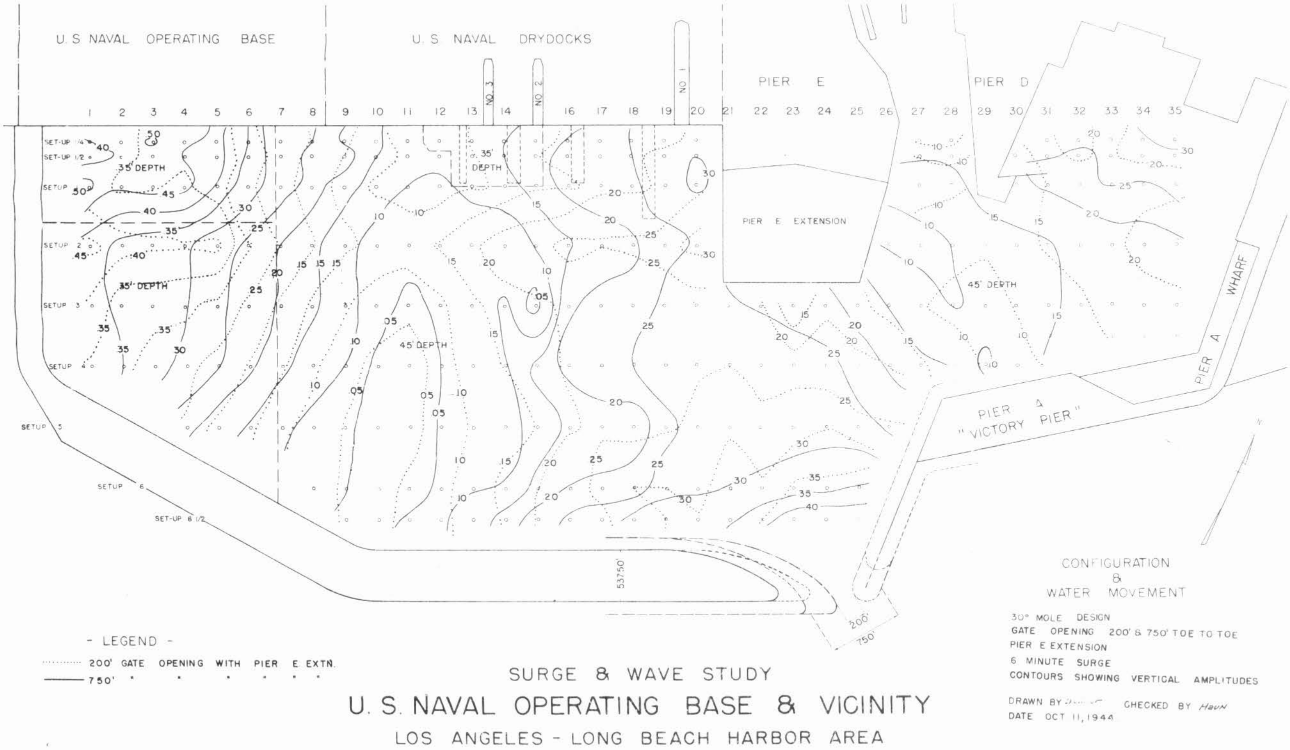


FIG. 124 VERTICAL MOVEMENT CAUSED BY 6 MINUTE SURGE
200 FT. GATE OPENING VS. 750 FT. GATE OPENING

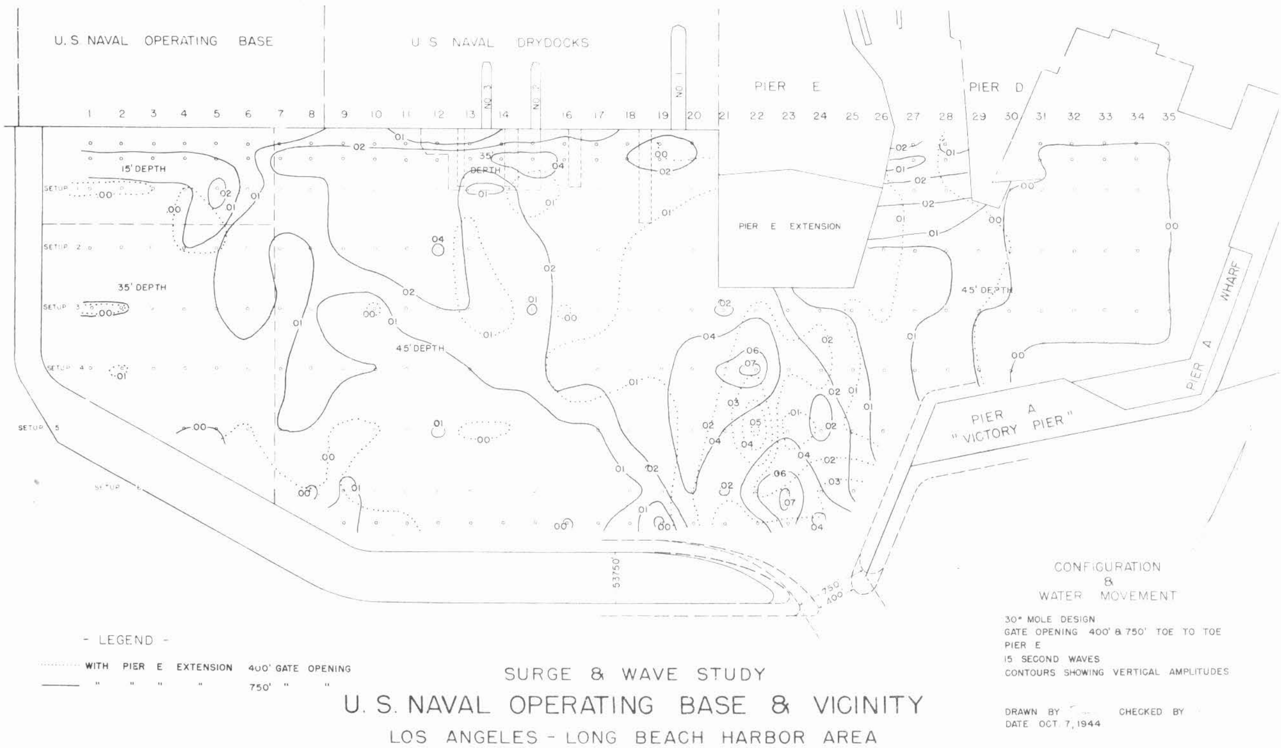


FIG. 125 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
400 FT. GATE OPENING VS. 750 FT. GATE OPENING

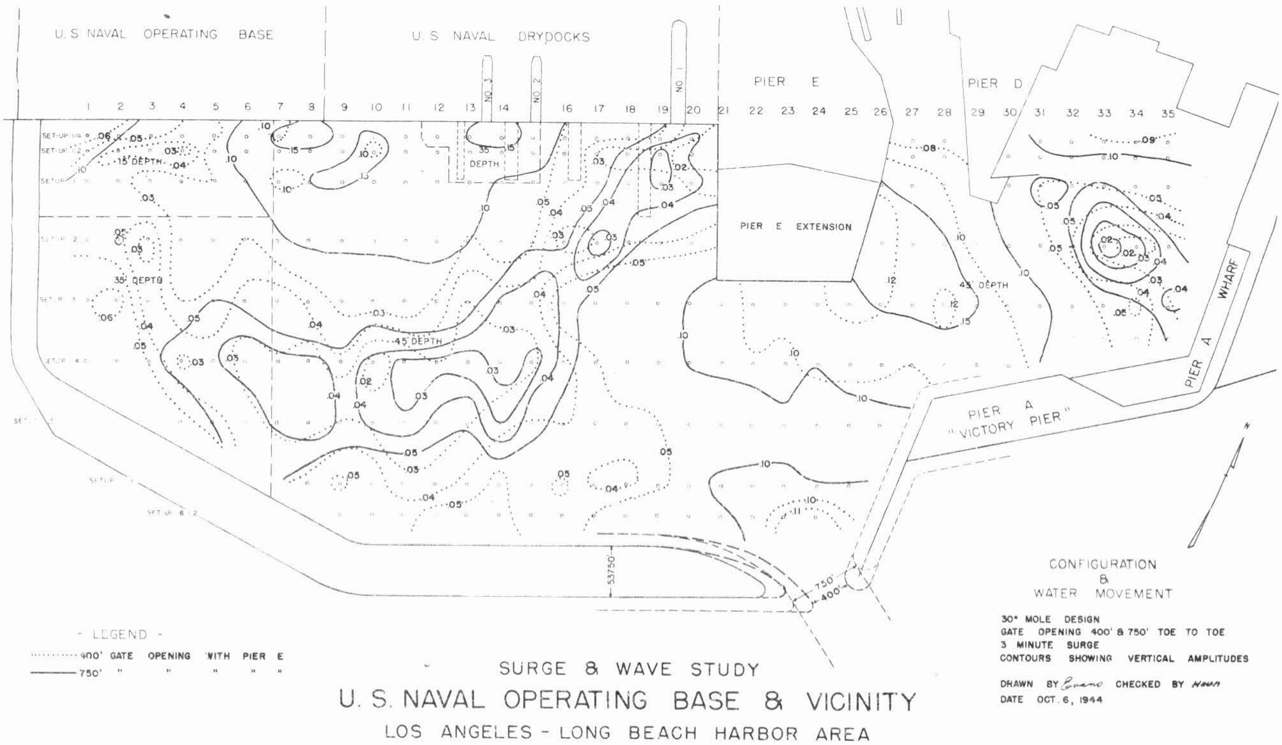


FIG. 126 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
400 FT. GATE OPENING VS. 750 FT. GATE OPENING

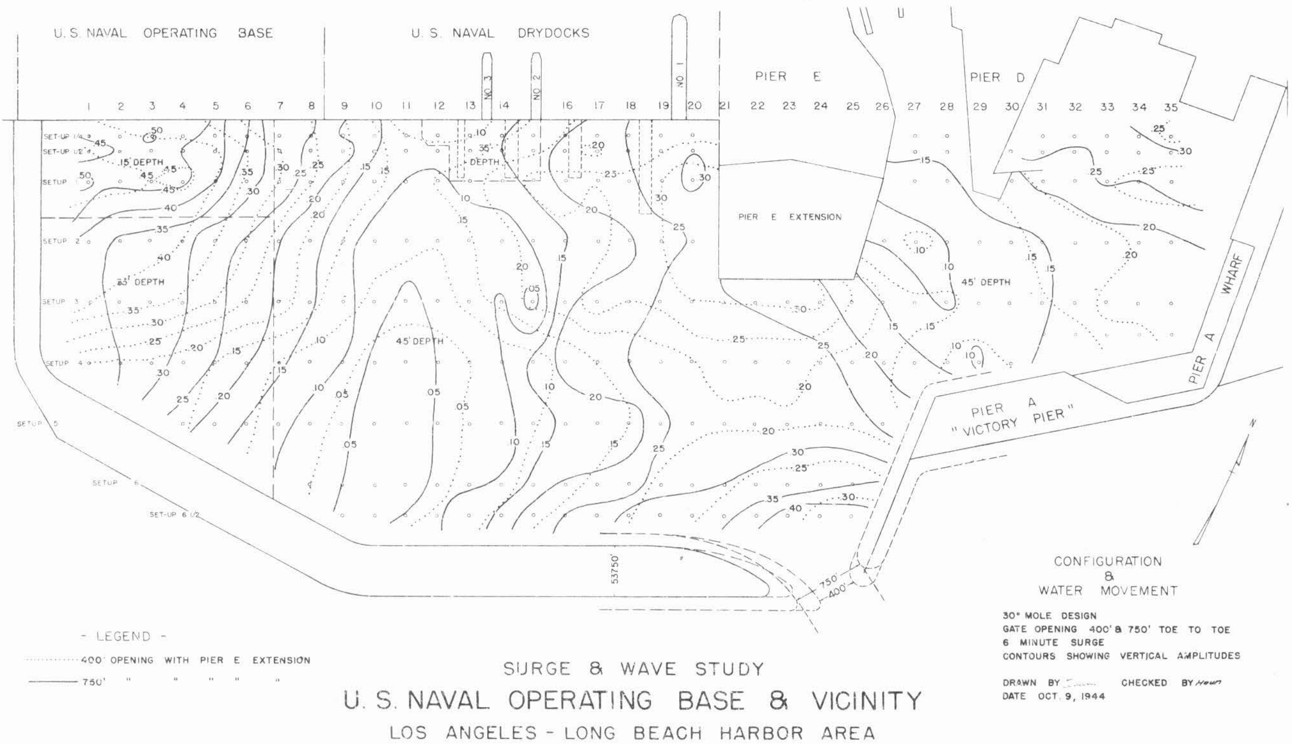


FIG. 127 VERTICAL MOVEMENT CAUSED BY 6 MINUTE SURGE
400 FT. GATE OPENING VS. 750 FT. GATE OPENING

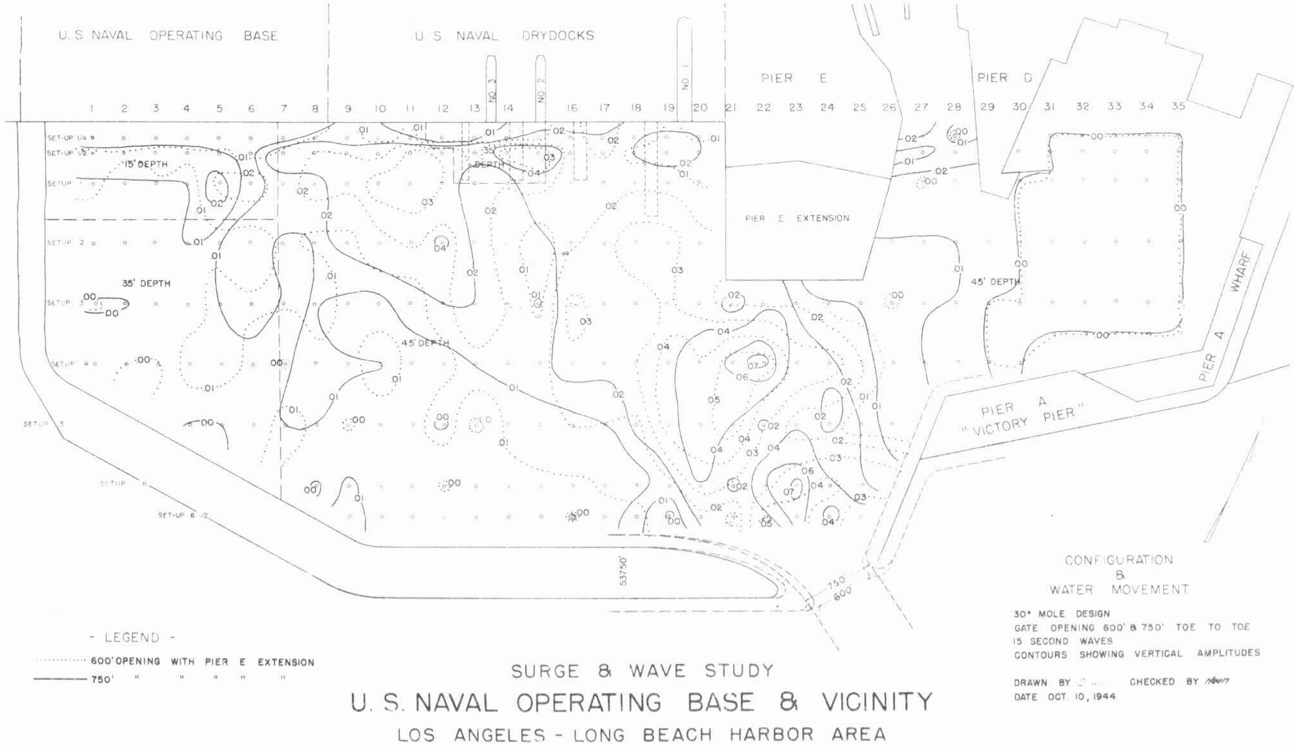


FIG. 128 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
600 FT. GATE OPENING VS. 750 FT. GATE OPENING

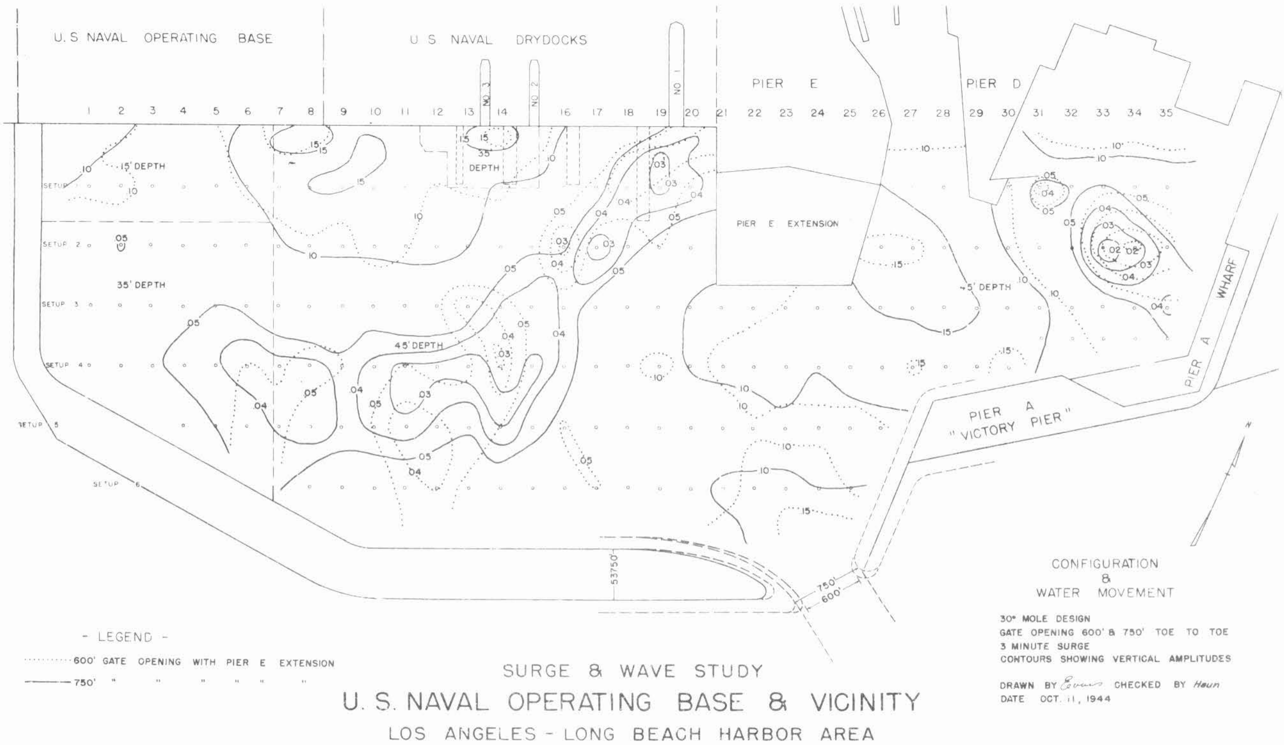


FIG. 129 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
600 FT. GATE OPENING VS. 750 FT. GATE OPENING

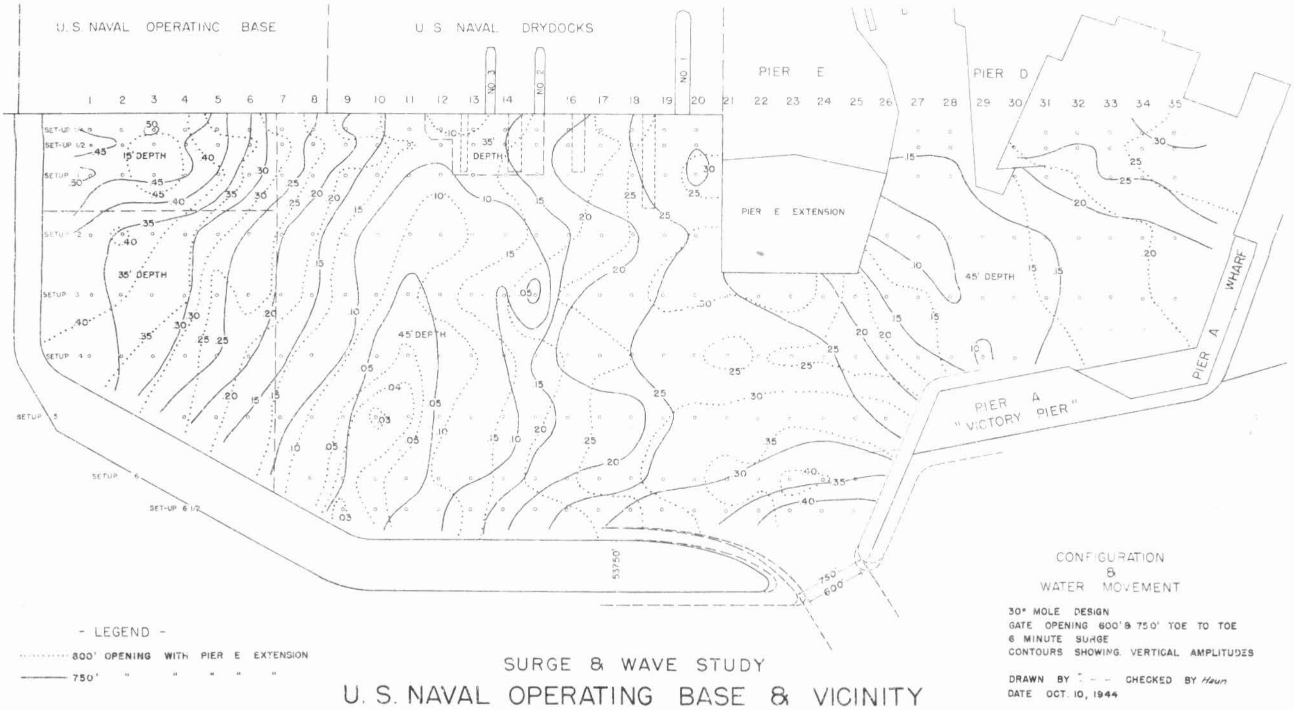


FIG. 130 VERTICAL MOVEMENT CAUSED BY 6 MINUTE SURGE
600 FT. GATE OPENING VS. 750 FT. GATE OPENING

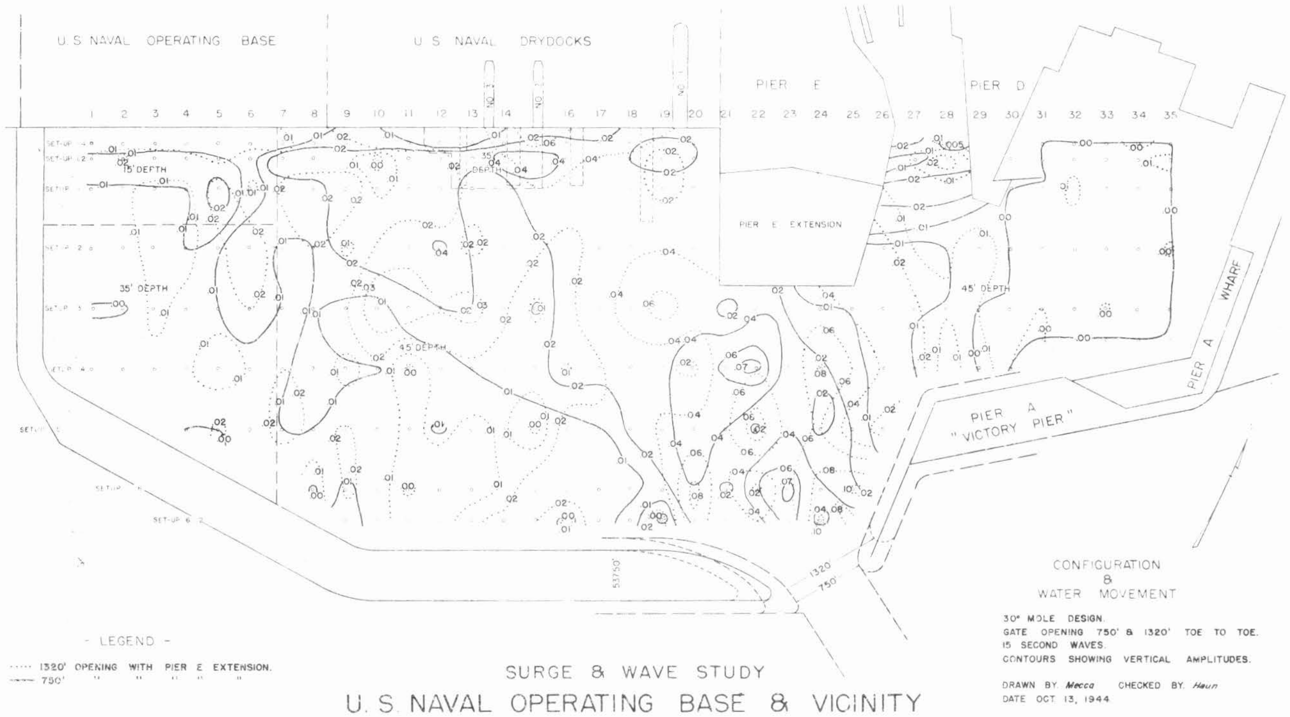


FIG. 131 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
750 FT. GATE OPENING VS. 1320 FT. GATE OPENING

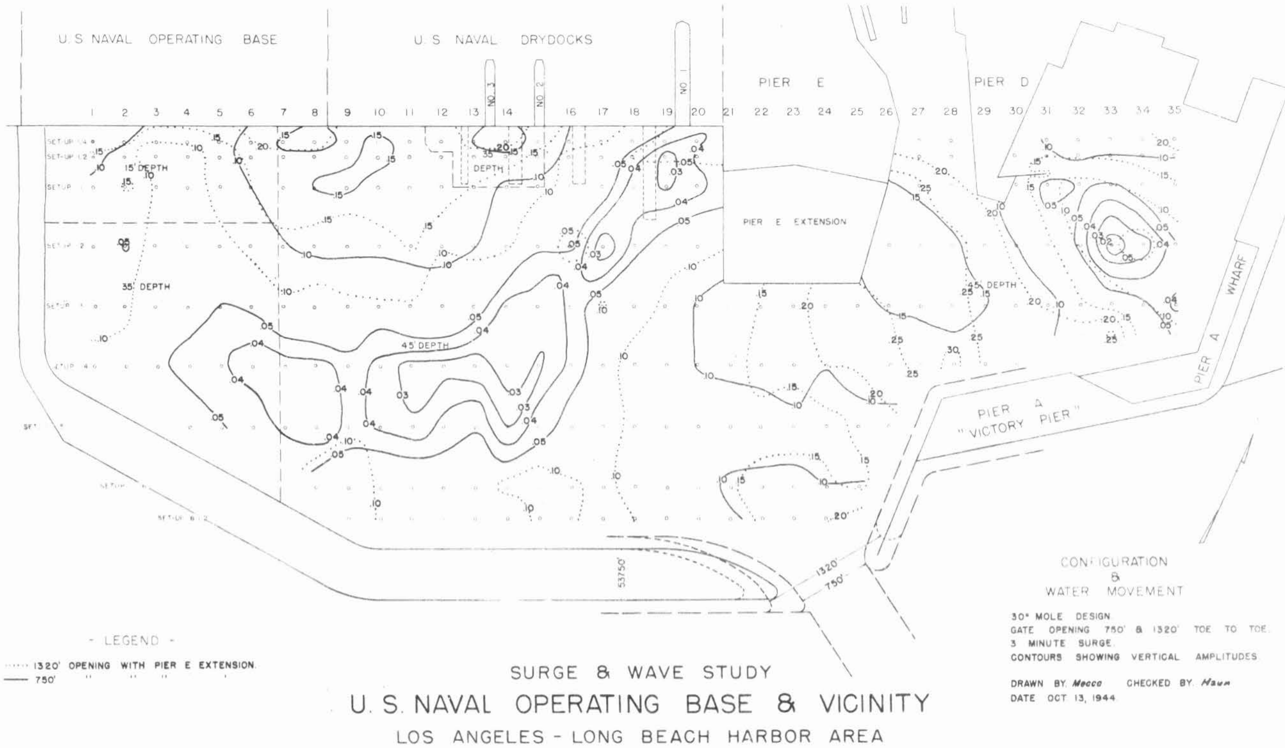


FIG. 132 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
750 FT. GATE OPENING VS. 1320 FT. GATE OPENING

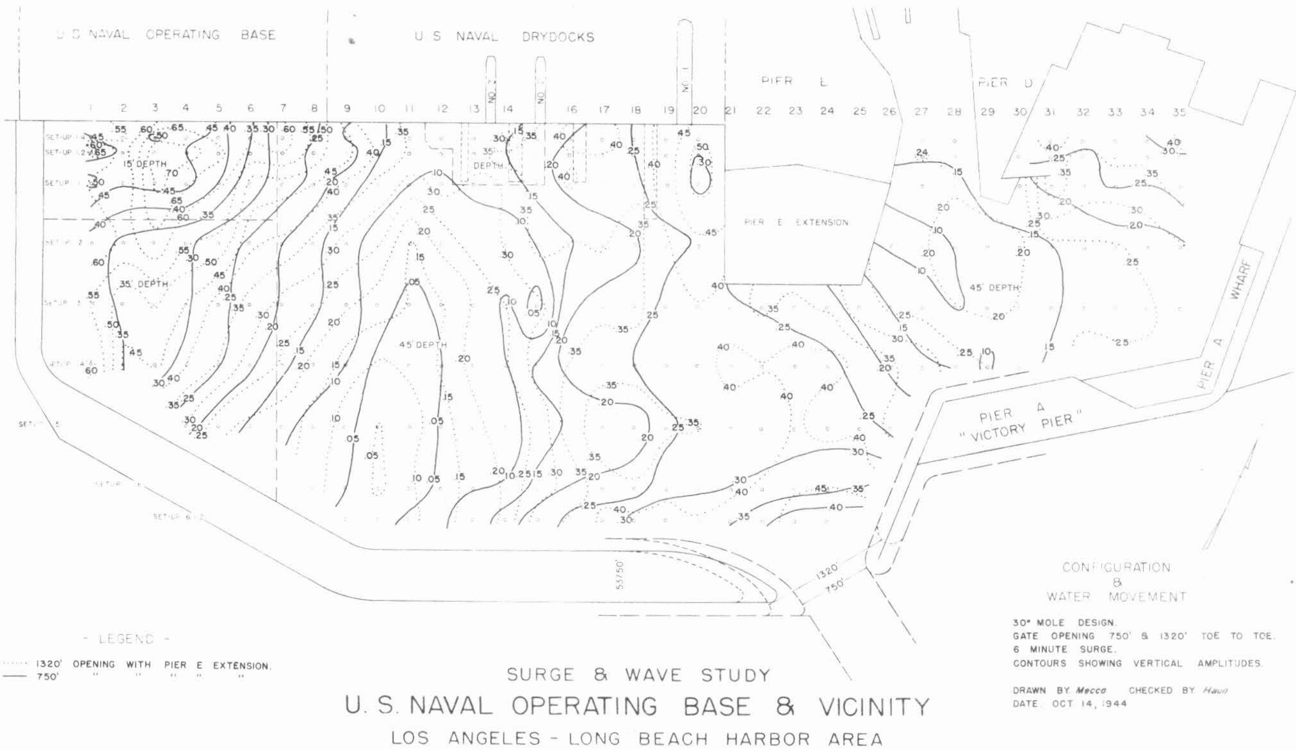


FIG. 133 VERTICAL MOVEMENT CAUSED BY 6 MINUTE SURGE
750 FT. GATE OPENING VS. 1320 FT. GATE OPENING

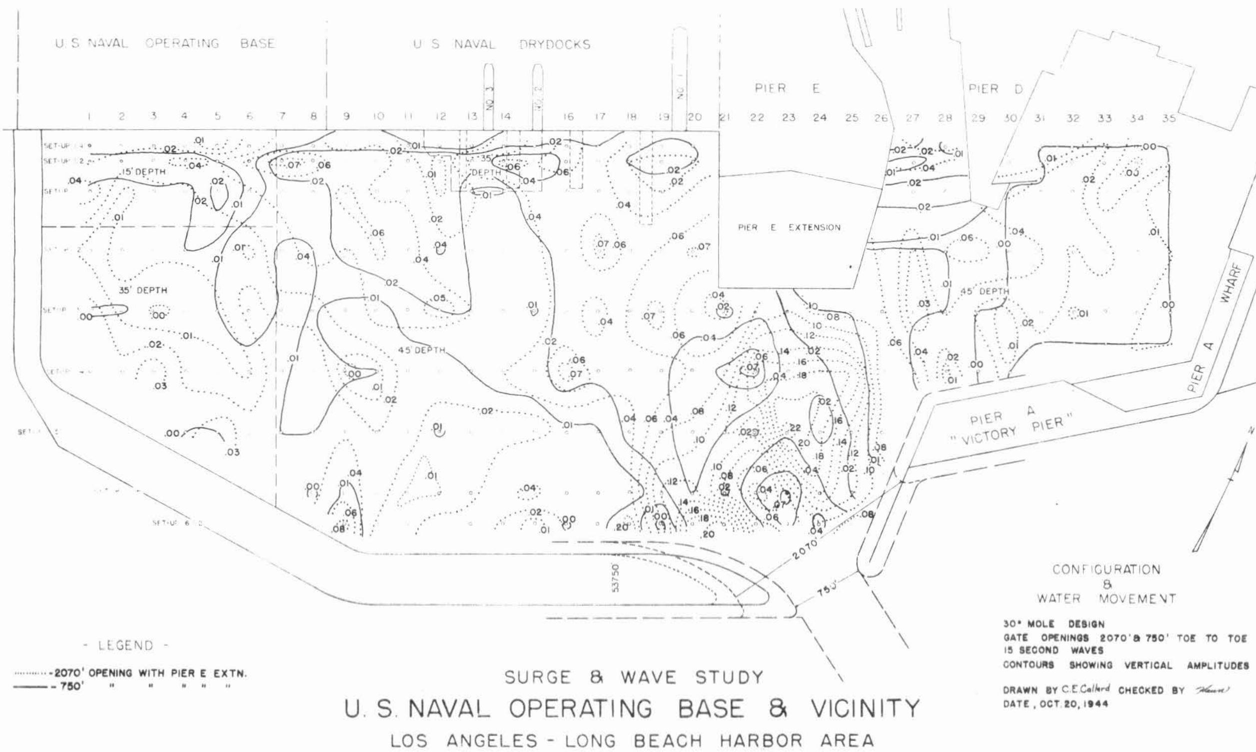


FIG. 134 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
750 FT. GATE OPENING VS. 2070 FT. GATE OPENING

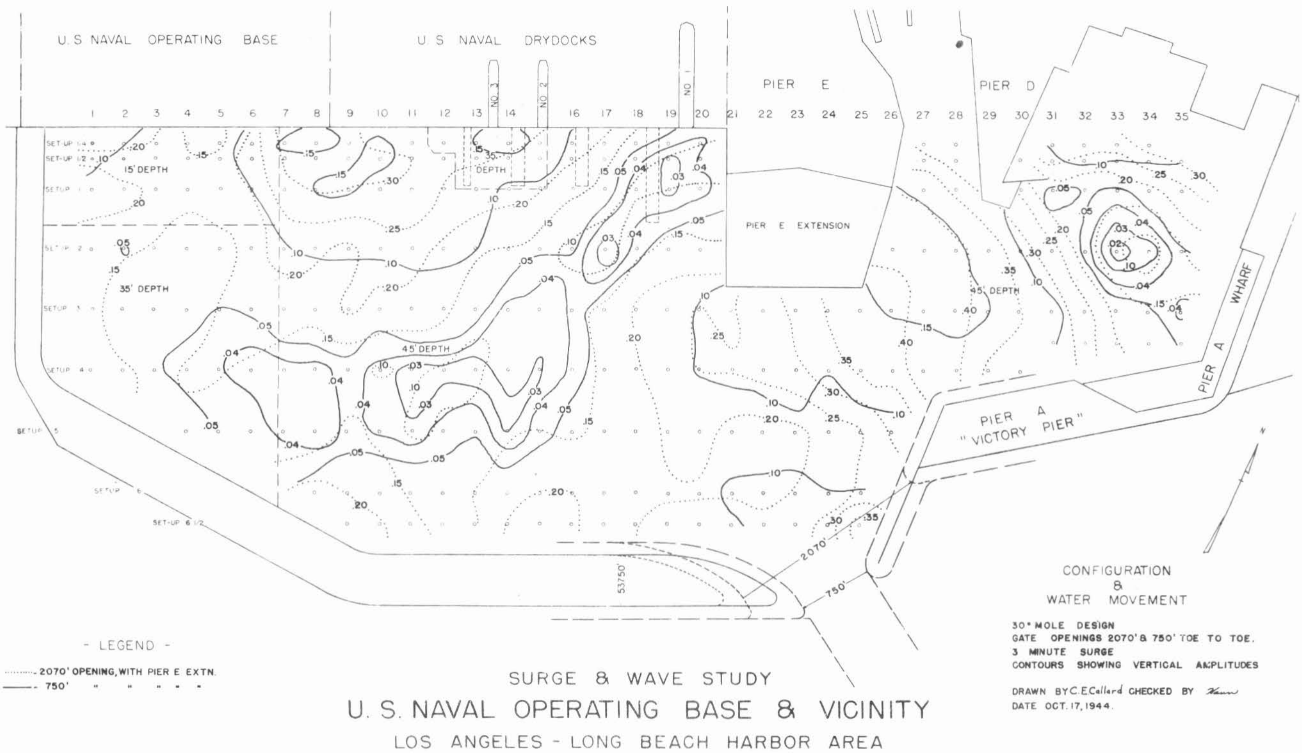


FIG. 135 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
750 FT. GATE OPENING VS. 2070 FT. GATE OPENING

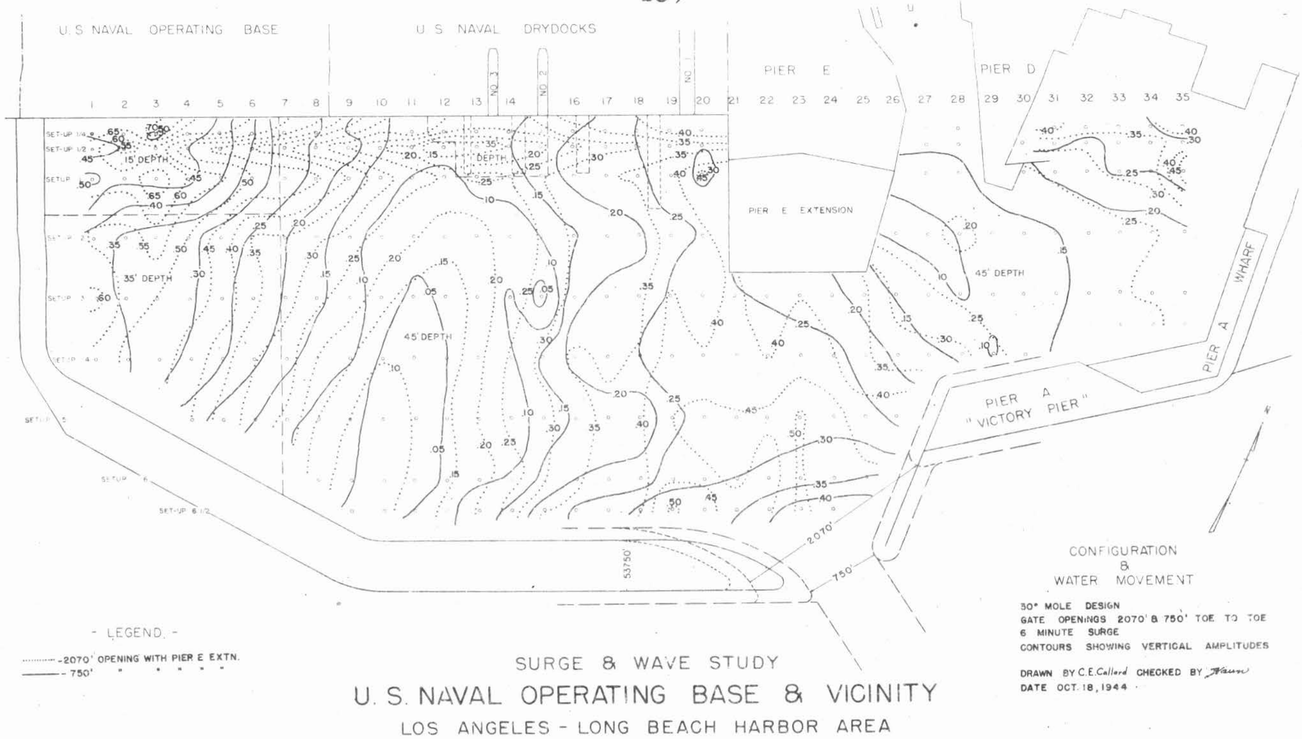
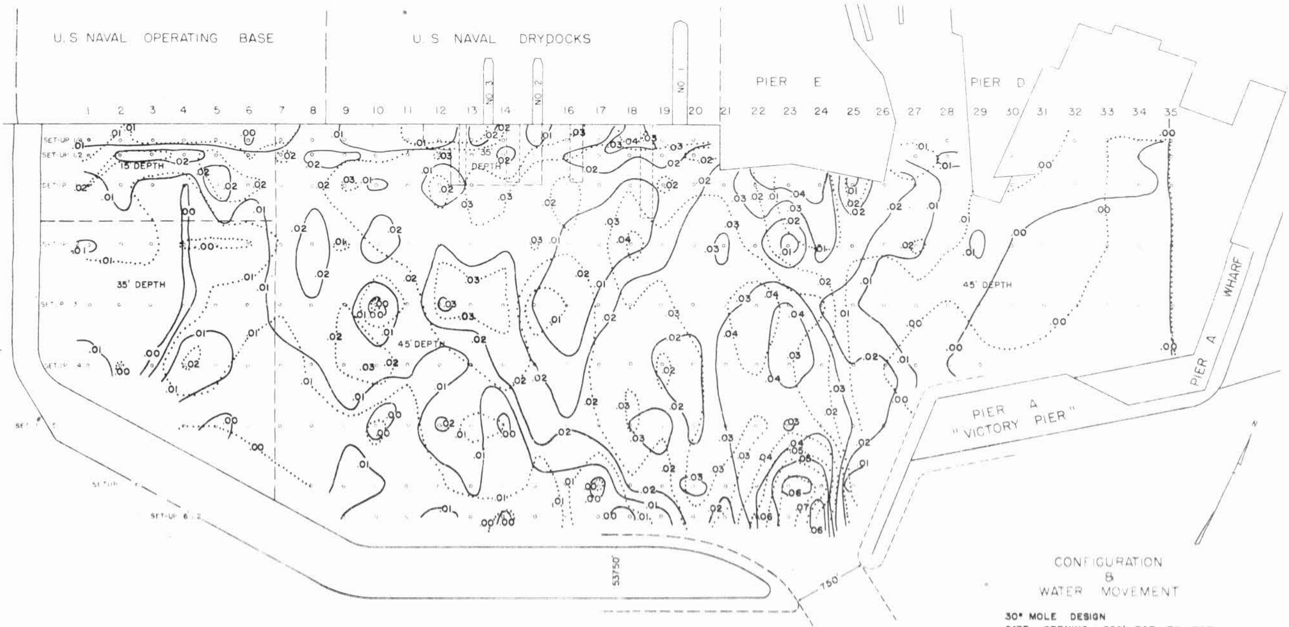


FIG. 136 VERTICAL MOVEMENT CAUSED BY 6 MINUTE SURGE
750 FT. GATE OPENING VS. 2070 FT. GATE OPENING

installation of a series of different structures that were proposed or authorized for installation within the basin. The purpose of this paragraph is to present a similar evaluation for the same series of structures installed in Model 3. In this case, however, all of the investigations were made with Pier A wharf in place, since it now is a part of the basin. However, comparison tests were run for the empty basin with and without Pier A wharf. The results are shown in the two-line comparison maps of Figures 137 to 139, inclusive, It will be observed that for the deepened basin the difference between the two conditions is much smaller than it was for the 35 ft. depth.

(2) Pier E extension. A comparison between the conditions in the basin with and without Pier E extension was made. The results for the three minute surge are shown in Figure 140. The movements produced by the three minute surge were considered to be more significant than those for either the fifteen second waves or the six minute surge. Therefore, the three minute surge has been used as the standard of comparison. It will be seen that the results for the deep basin of Model 3 confirm those of Model 2 that Pier E extension reduces the amount of motion in the basin. This reduction is quite appreciable in the drydock area.

(3) Marginal wharf and mole piers. Figure 141 shows the conditions within the basin with and without Pier E and the marginal wharf and mole piers on the parallel leg of the mole. The presence of the mole piers again produces a decrease in the vertical amplitude of motion.

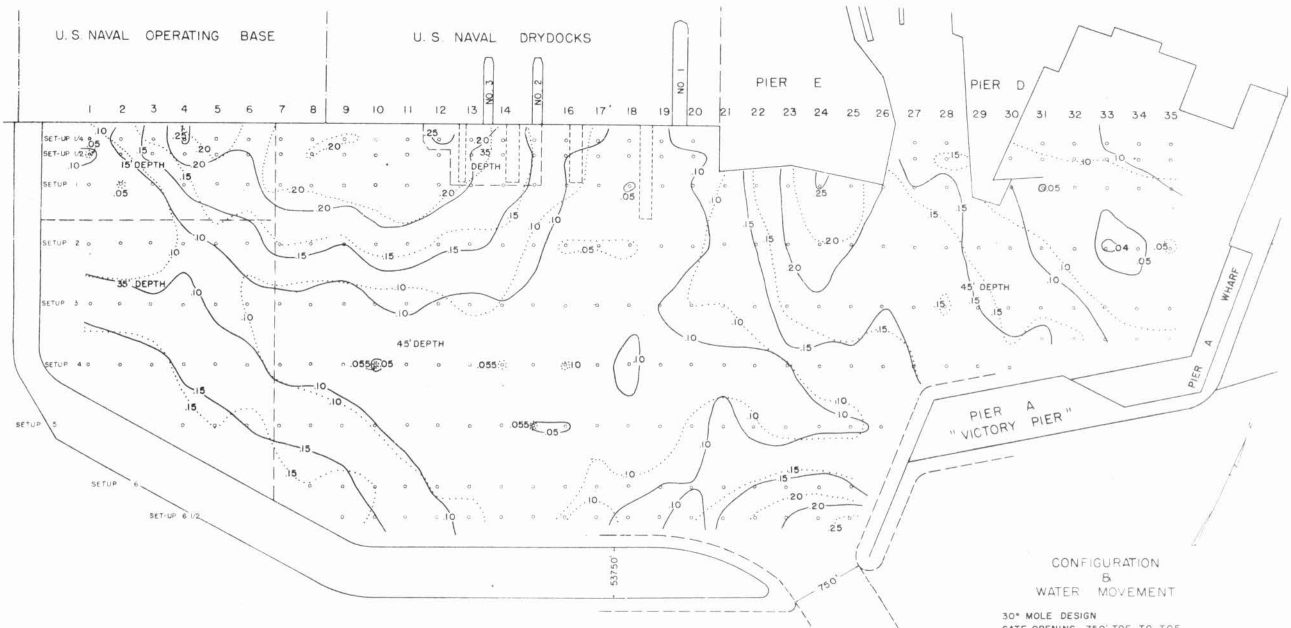


- LEGEND -
WITHOUT PIER A WHARF
WITH " A "

CONFIGURATION & WATER MOVEMENT
30° MOLE DESIGN
GATE OPENING 750' TOE TO TOE
15 SECOND WAVES
CONTOURS SHOWING VERTICAL AMPLITUDES
DRAWN BY Weimer CHECKED BY HAUN
DATE: OCT. 16, 1944

SURGE & WAVE STUDY
U. S. NAVAL OPERATING BASE & VICINITY
LOS ANGELES - LONG BEACH HARBOR AREA

FIG. 137 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES WITH AND WITHOUT PIER A WHARF



- LEGEND -
WITHOUT PIER A WHARF
WITH " "

CONFIGURATION & WATER MOVEMENT
30° MOLE DESIGN
GATE OPENING 750' TOE TO TOE
3 MINUTE SURGE
CONTOURS SHOWING VERTICAL AMPLITUDES
DRAWN BY A.C.S. CHECKED BY
DATE NOV. 18, 1944

SURGE & WAVE STUDY
U. S. NAVAL OPERATING BASE & VICINITY
LOS ANGELES - LONG BEACH HARBOR AREA

FIG. 138 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT PIER A WHARF

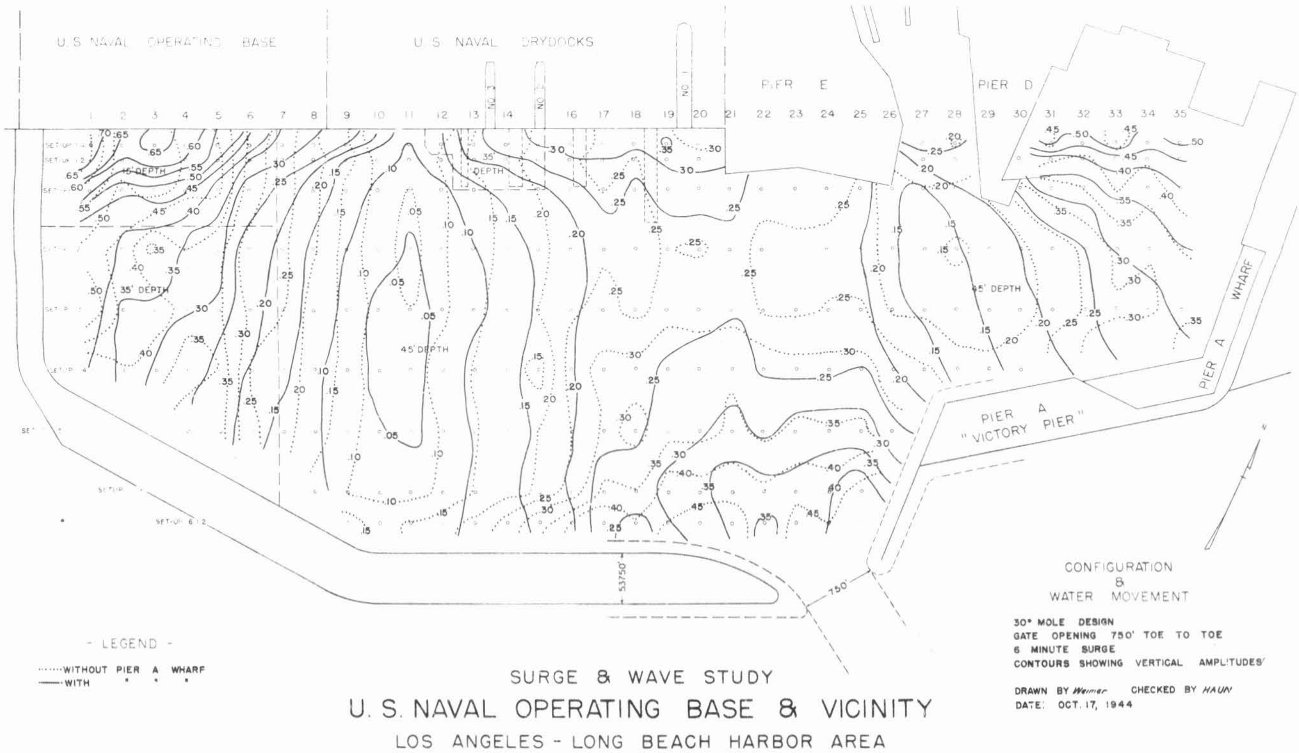


FIG. 139 VERTICAL MOVEMENT CAUSED BY 6 MINUTE SURGE WITH AND WITHOUT PIER A WHARF

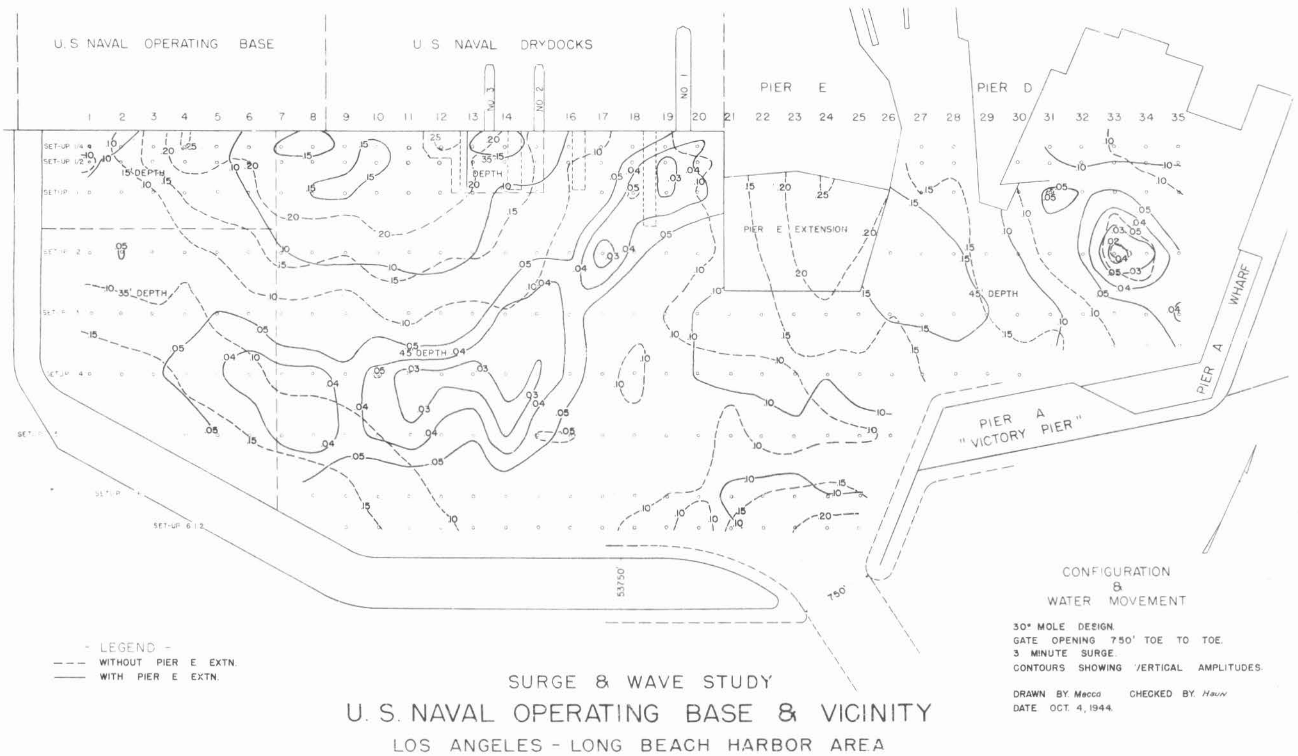


FIG. 140 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT PIER E EXTENSION

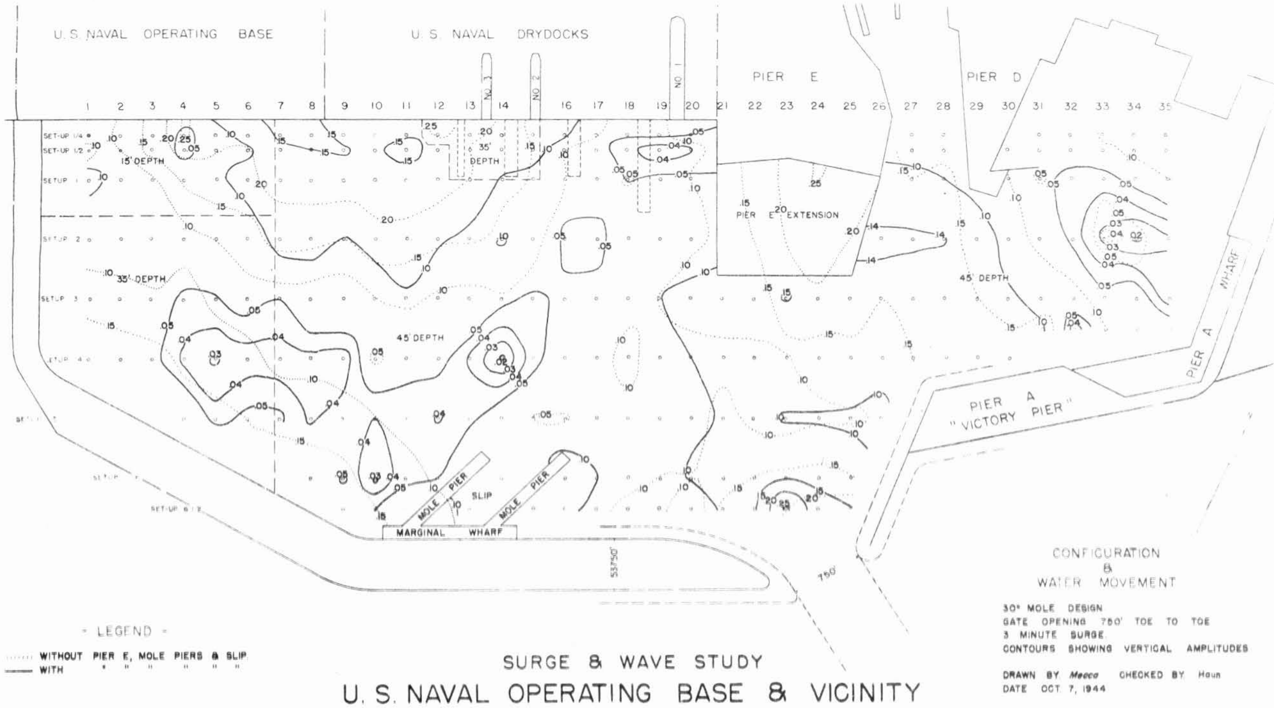


FIG. 141 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT MARGINAL WHARF AND MOLE PIERS

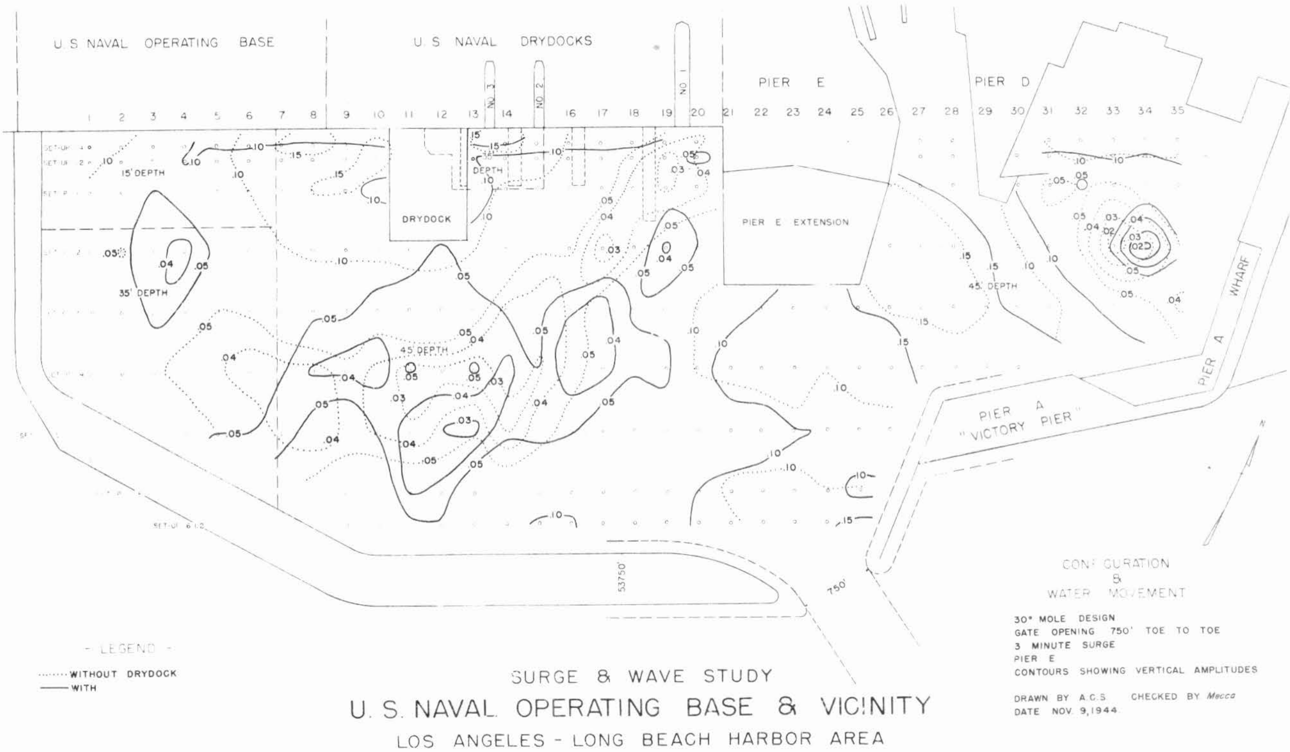


FIG. 142 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT DRYDOCK FILL

(4) Drydock fill. Figure 142 gives the comparison with and without the presence of the proposed drydock, whose installation was contemplated to the west of Pier 3. In this case it will be observed that there is little change in pattern in the general area of the basin, although the vertical motion has been reduced locally to the west of the drydock by its installation. However, in the remainder of the basin, if there is any significant difference, it seems that the motion with the drydock in place is slightly greater than without.

(5) Mole piers combined with Pier E extension and drydock fill. Figure 143 shows the motion in the basin with and without the mole piers and marginal wharf on the parallel leg of the mole, but with both Pier E extension and drydocks in place. Again, it will be noted that the installation of the mole piers and marginal wharf improves conditions in the basin. However, with drydocks in place there appears to be a region of rather high vertical amplitude at the end of the east mole pier which might interfere with its effectiveness.

(6) Use of diagonal leg of mole for piers. Figure 144 shows the conditions that result if the mole piers and marginal wharf are moved to the center of the diagonal leg. It will be observed that in this location the mole piers seem to produce less reduction in motion on the basin as a whole than they do when they are located on the parallel arm. However, conditions at the mole piers themselves have been improved.

(7) Mole piers with protective wharf. Figure 145 shows the result of adding a protective wharf on the parallel leg. It will be observed that the general conditions within the basin have been improved appreciably by this protective wharf and the conditions at the mole piers and marginal wharf are very satisfactory. It should be remembered that the mole piers and the protective wharf are all assumed to be of opaque, i.e., solid construction. Open pile piers in these locations would have no effect upon the motion in the basin.

(8) Opaque piers. A study was made of the effect of substituting opaque piers for the open piers #1 to #4 that are now in existence. Figure 146 shows the basin conditions with and without these opaque piers. It will be observed that with the opaque construction conditions in the basin are improved considerably. Docking conditions also are improved for Piers 2, 3 and 4, but with this configuration apparently Pier 1 is, if anything, somewhat worse than it is with the transparent piers.

(9) Comparison of basin with and without all proposed structures. In the previous discussion the effect of additional structures within the basin has been presented by comparing conditions after the addition of each structure with those that existed without it. In this manner the basin has been investigated for the initial condition of no structures in the area and then studied step by step as structures were added until a large part of the shoreline of the basin was developed to its maximum possibilities. It

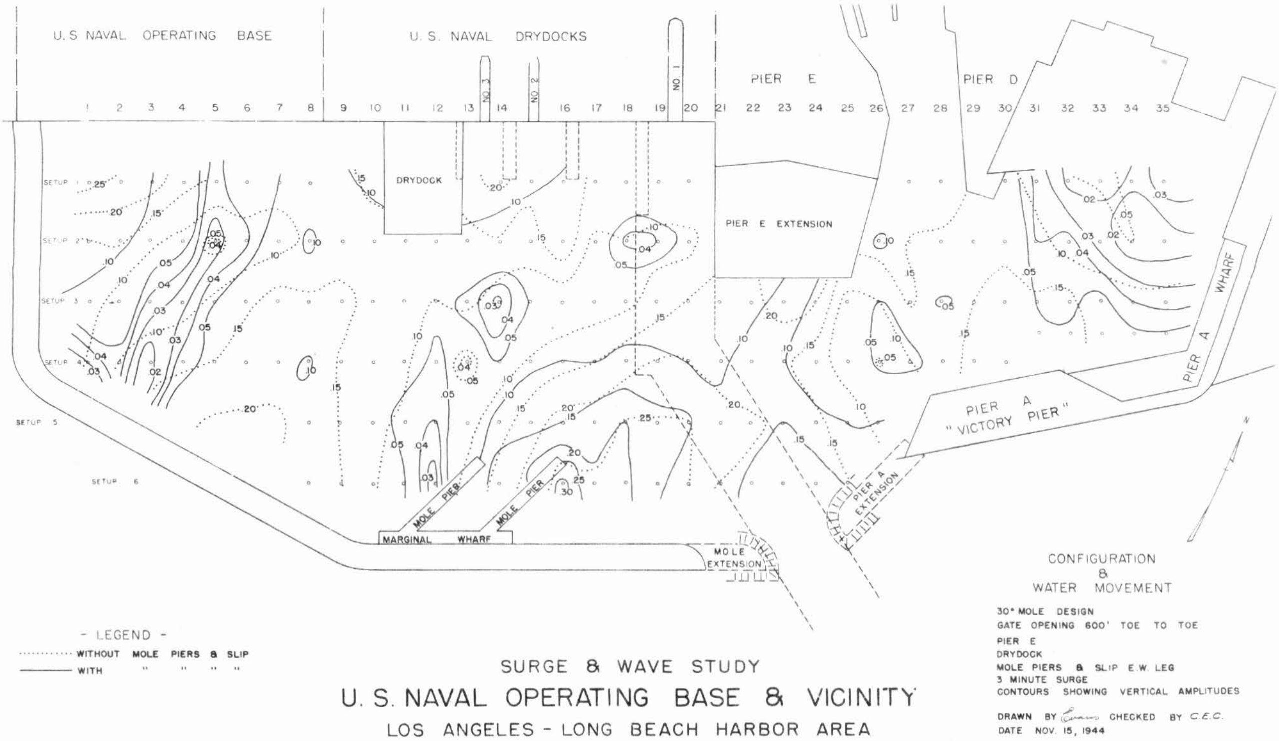


FIG. 143 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT MARGINAL WHARF AND MOLE PIERS WITH PIER E EXTENSION AND DRYDOCK EXTENSION

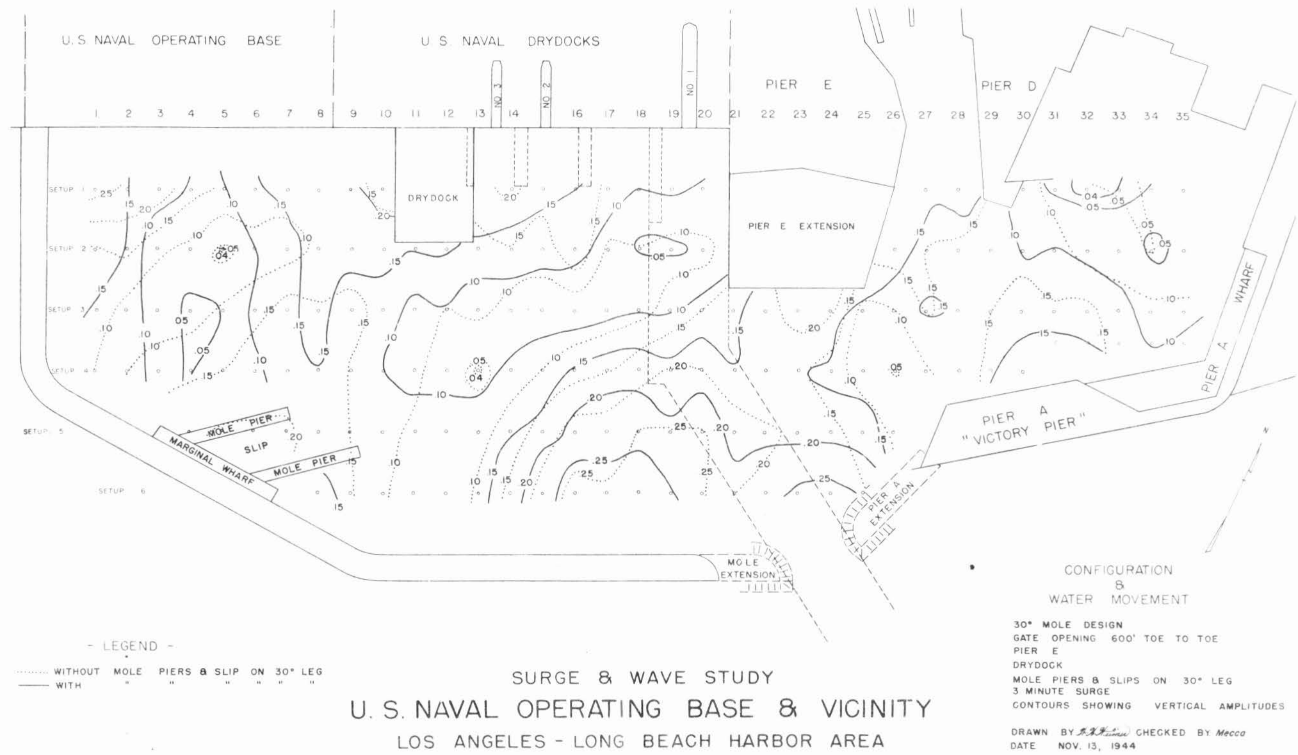


FIG. 144 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT MARGINAL WHARF AND MOLE PIERS ON 30° DIAGONAL

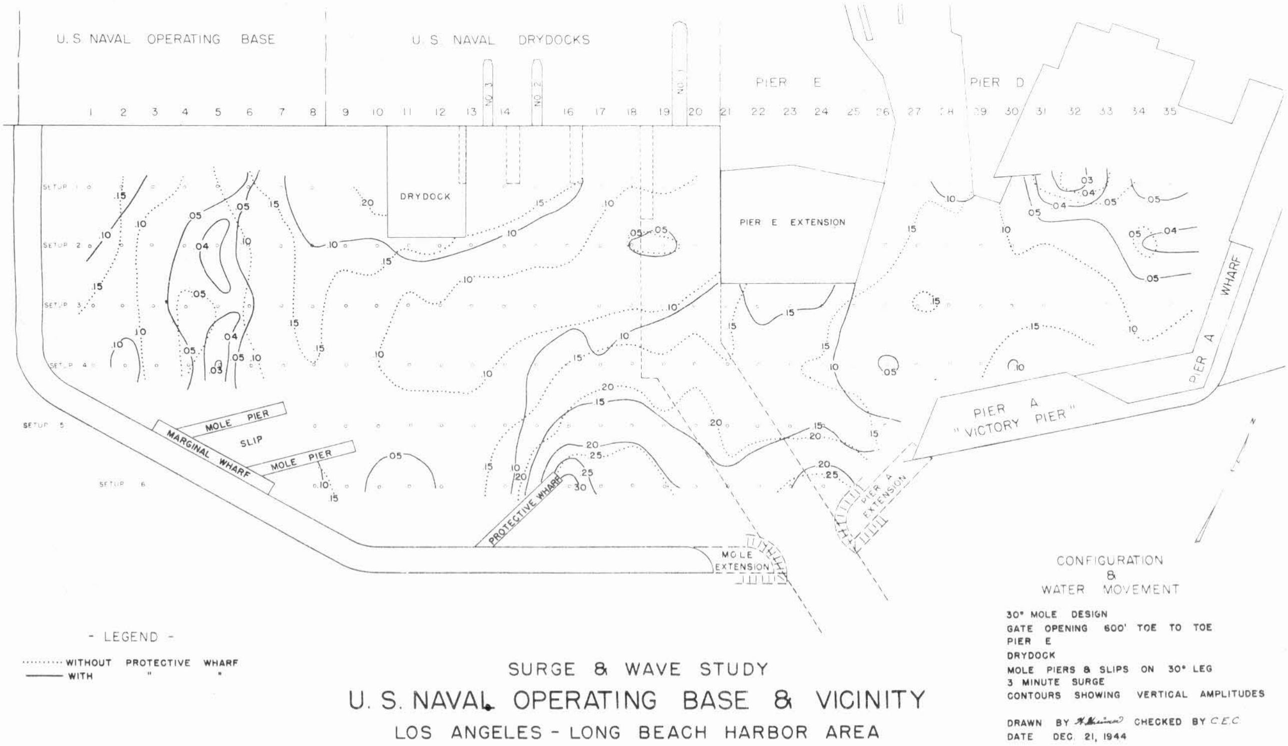


FIG. 145 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT PROTECTIVE WHARF

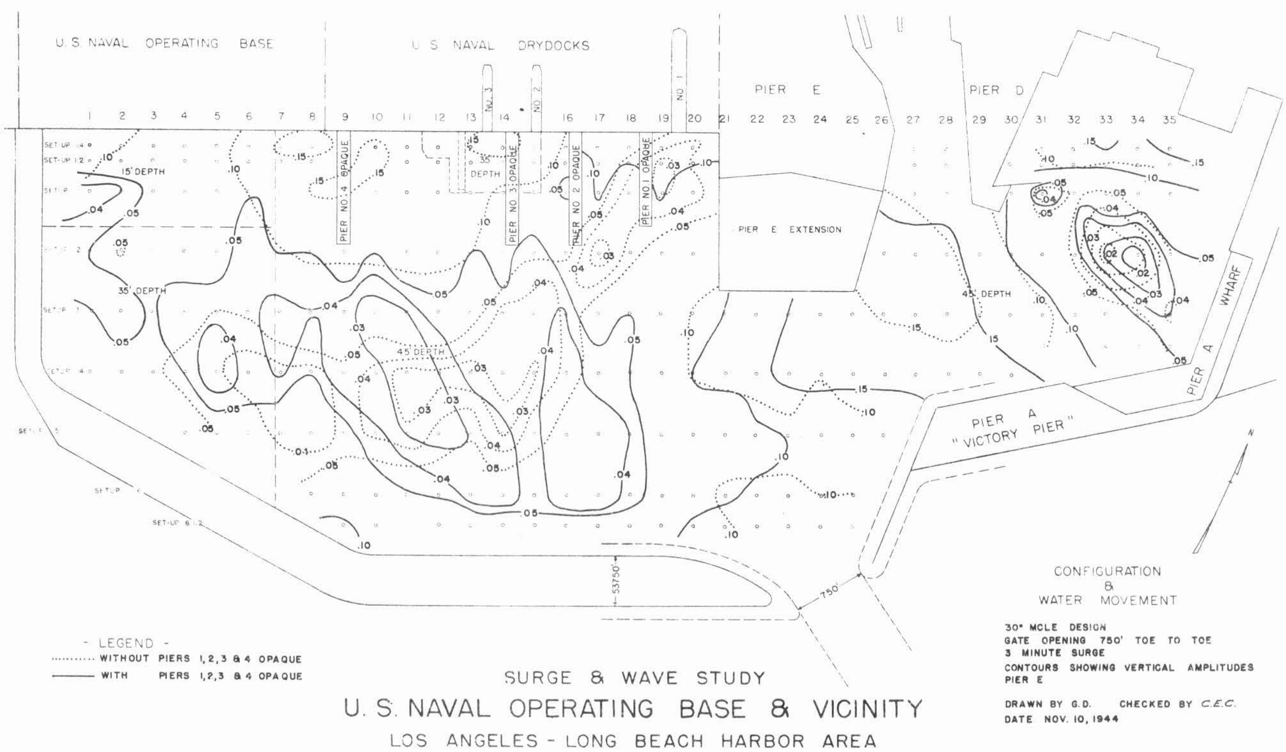


FIG. 146 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE WITH AND WITHOUT OPAQUE PIERS #1, #2, #3 AND #4

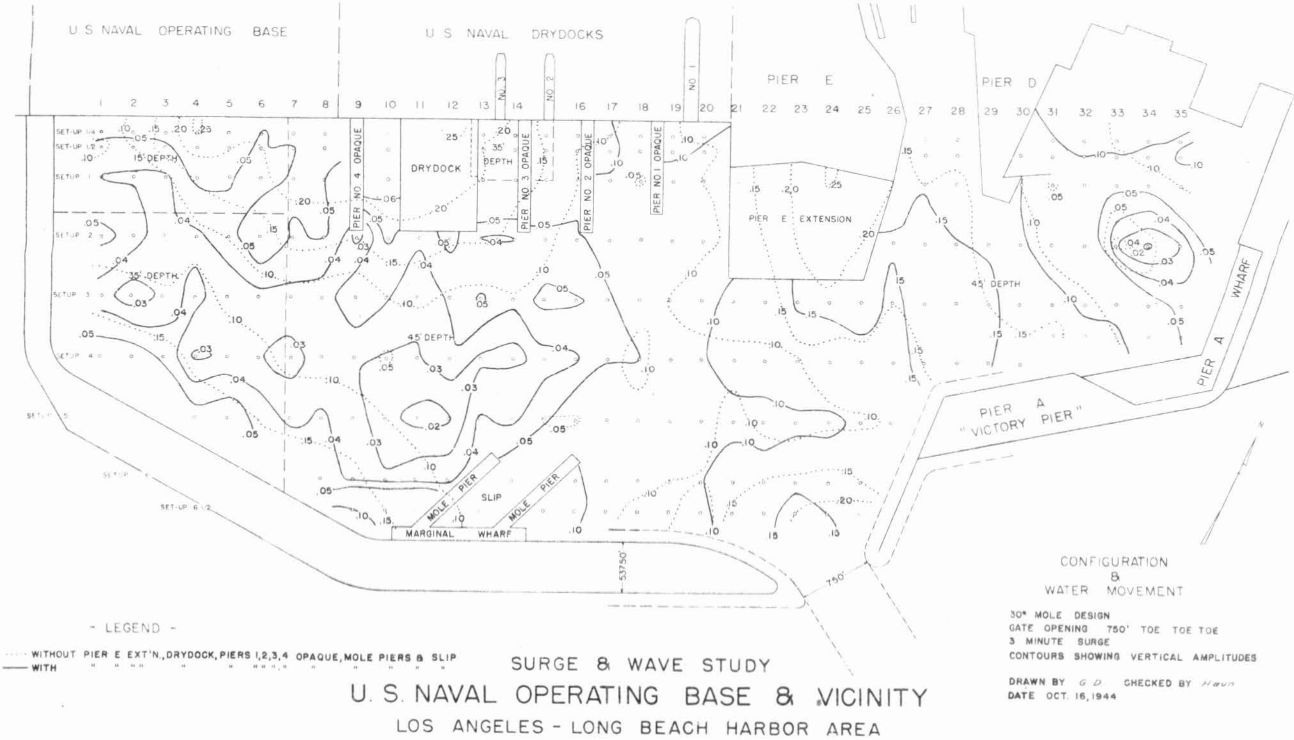


FIG. 147 VERTICAL MOVEMENT CAUSED BY 3 MINUTE SURGE
BASIN WITH AND WITHOUT ADDITIONAL STRUCTURES

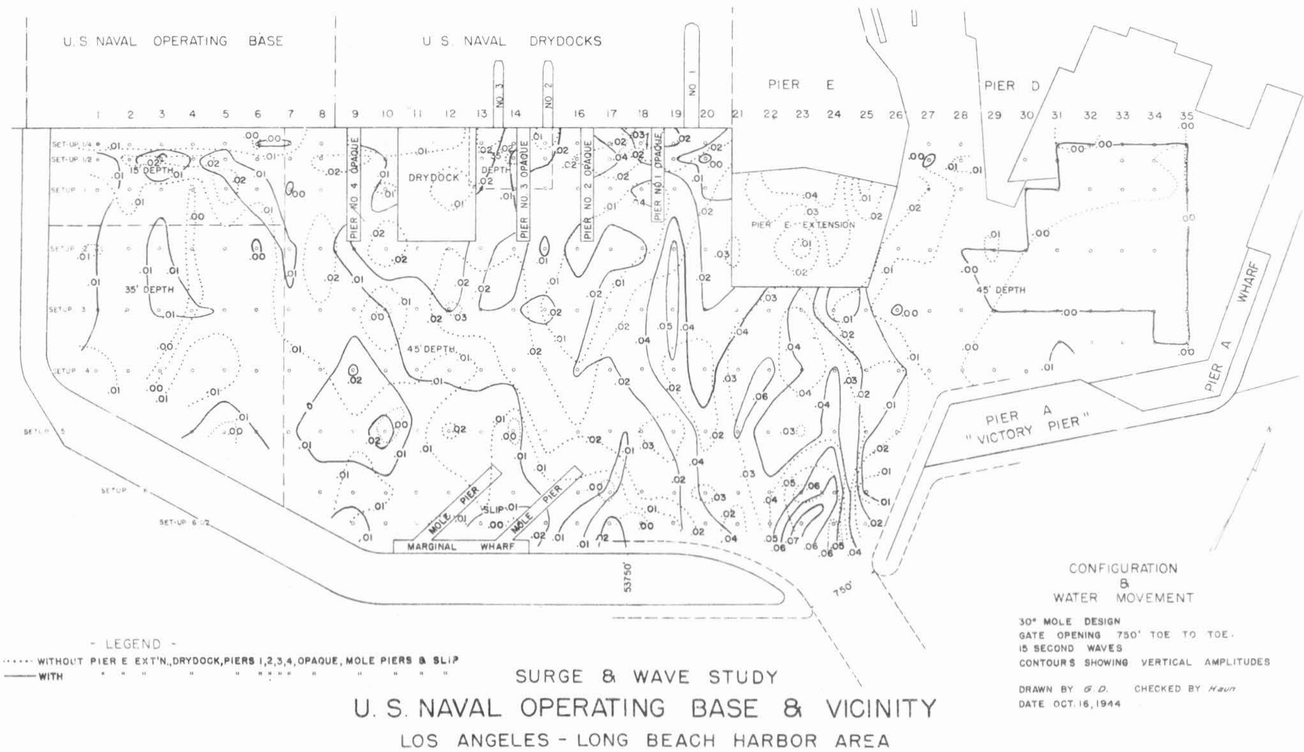


FIG. 148 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES
BASIN WITH AND WITHOUT ADDITIONAL STRUCTURES

is instructive to examine the over-all effect of the additional structures within the basin by comparing directly the water movements when the entire group is in place with those existing with the basin empty. Figure 147 shows this comparison for the three minute surge train. It will be observed that at nearly every point within the basin, conditions have been improved considerably by the presence of the additional structures. This is true in the area to the west of the gate, which includes all the Naval Drydocks and Naval Operating Base area. However, in the Long Beach harbor area, which lies to the east of the mole gate, very little effect can be noticed, although what little change occurs is an improvement. Figure 148 gives a similar comparison, but this time for the fifteen second wave trains. With these waves it will be noted that the addition of the structures within the basin does little more than modify the details of the standing wave pattern; the range of vertical amplitudes remains about the same. Figure 149 gives a third comparison, this time for the six minute surge. Here again, it will be observed that addition of the structures has served mainly to modify the standing wave pattern, increasing the vertical amplitudes of the water motion in some areas and decreasing it in others. The northwest corner of the basin consistently shows the maximum motion. In this area the addition

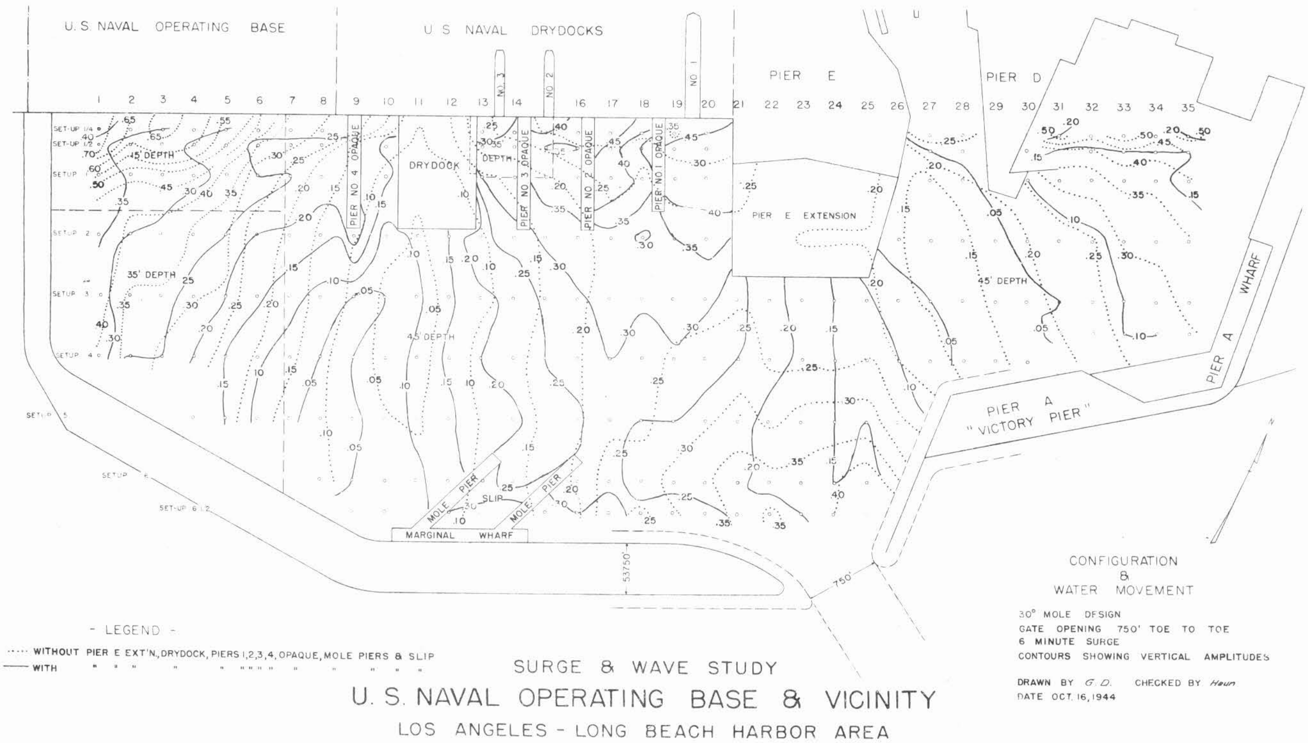


FIG. 149 VERTICAL MOVEMENT CAUSED BY 6 MINUTE SURGE BASIN WITH AND WITHOUT ADDITIONAL STRUCTURES

of the structures has produced an appreciable diminution in the amplitude. Furthermore, in the Long Beach harbor area, the vertical motion due to the six minute surge, has been decreased to approximately half amplitude by the addition of the structures in the basin. However, along the drydocks waterfront, the amplitude, if anything, is increased by the presence of the structures. It is striking to observe that, although in the outer harbor area the amplitudes of the exciting wave trains are approximately equal for the three different wave periods, the motion within the basin for the six minute surges is, on the average, several times greater than it is with the three minute surges; while the motion due to the fifteen second waves is much lower than that due to the three minute surges.

(f) Horizontal motion in the basin. In the course of this section of the study observations were made of the horizontal water movements by means of reflectors, using the same technique employed with similar studies on Model 2. Figures 150 to 155, inclusive, show a summary of the results of these studies for the three different wave trains. However, with Model 3, investigations were made only for the 2070 ft., the 1320 ft., and the 750 ft. gate openings. These Figures are comparable to Figures 96 and 97 of the results of Model 2. They can be used to obtain several cross-comparisons. The three photographs in each figure show the movements produced by the three standard wave trains. Figures 150, 151, and 152 demonstrate the effect of changing the width of the gate on the behaviour of the empty basin. Figures 153, 154, and 155 show, for the 750 ft. gate, the results of adding piers, dry-docks, and slips to the basin. During the taking of these reflector shot photographs it was found desirable to indicate the direction of the motion of each reflector and also to obtain intermittent time measurements, so as to detect any changes in velocity as a reflector moved about. This was accomplished by turning the main light source on and off in accordance with a prearranged schedule. These breaks may be seen in the traces of some of the more rapidly moving reflectors in the three minute and six minute surge pictures.

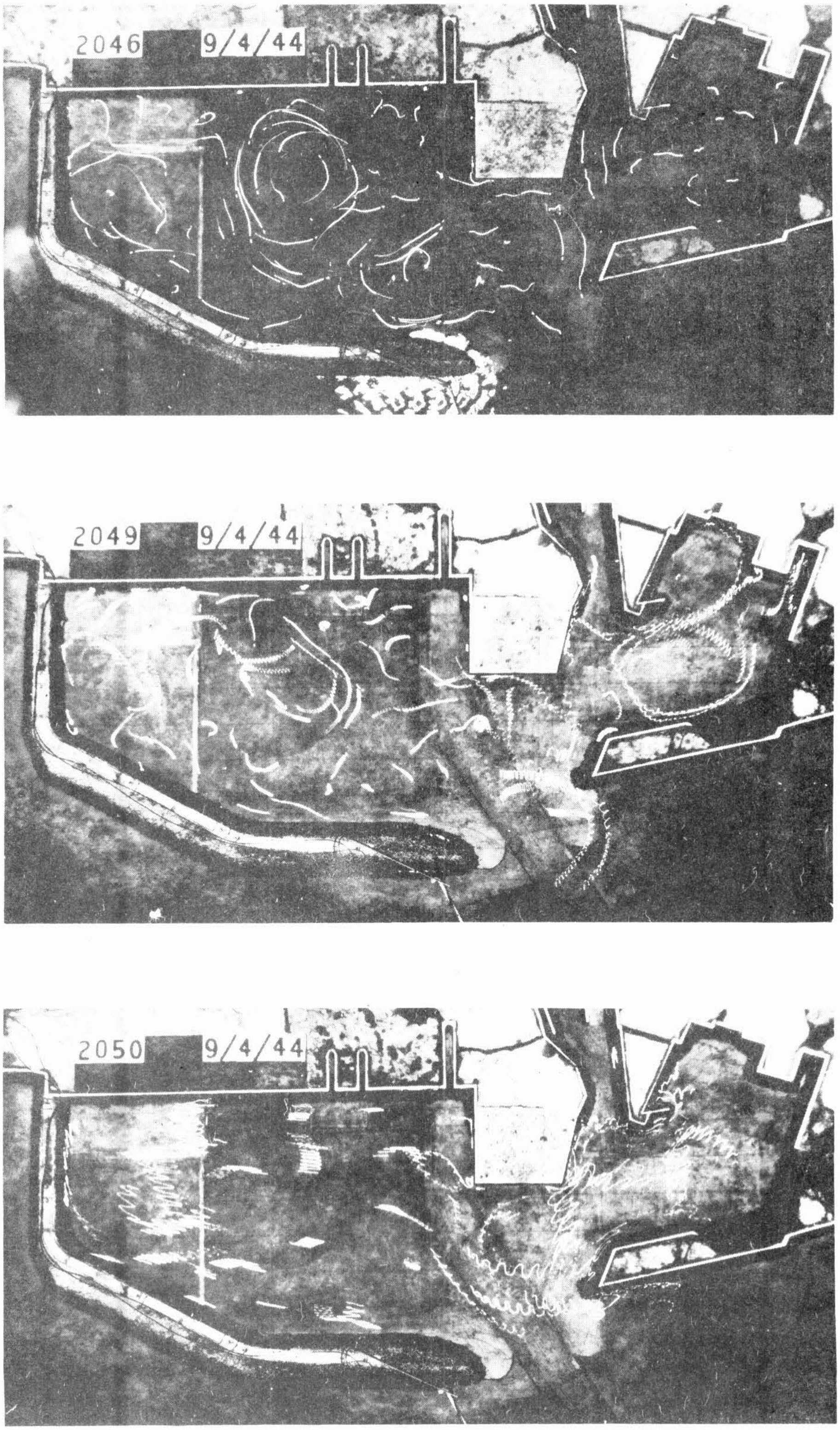
(1) Relative reliability of oscillating motions vs. drift currents. In Section IV-C it was pointed out that the existence of a standing wave pattern of vertical motion necessitated a corresponding pattern of horizontal motion, since it is necessary that the water flow back and forth to form the alternating crests and troughs. However, when the standing wave pattern is the result of the addition of a rather large series of reflected waves coming from a complicated boundary, as in the case of the present basin, it is very difficult to predict the horizontal motions from the observation of the vertical motions of the water surface. This is because the direction, amplitude and phase of all the individual wave trains making up the over-all pattern are not known precisely at any given location. As was previously pointed out, the use of the reflector floats was developed to study these horizontal motions. In observing the photographic records of these reflector studies, two different types of horizontal motion can be distinguished. Consider, for example, Figure 156. This shows the horizontal movement when the basin is excited by a six minute surge train. The lines of motion of the reflectors show clearly a cyclic motion having the same six minute period as that of the exciting train. However, this motion is superimposed upon what may be called a drift, or steady current, which is apparently a

by-product of the surge. For any given location the cyclic motion is characteristic and reproducible. The drift or current is not so reproducible. This is probably due to a combination of effects. In the first place, the current velocity is very low, which means that it is the result of a very small force. It is, therefore, easily affected even by the lightest breeze. In the second place, the current pattern is of large scale with respect to the dimensions of the basin, and once established it persists for a long period. Thus, it is very difficult to be sure that the currents produced by one test run are completely damped out before a second run is begun.

In order to investigate the building up and the damping out of the drift current, a series of special test runs was made in the following manner. All test activities were suspended for several hours and the basin was allowed to come to rest. The surge machine was then started, thus beginning the test. Two minutes later the first reflector photograph was taken. The second photograph was taken five minutes after the surge began and photographs were then taken at five minute intervals for a total of eighty minutes from the start of the test. The surge machine was stopped at the end of forty minutes. Thus, the first half of the time was available for the currents to build up and the second half for them to decay. To establish the directions of the motions recorded by the reflectors, the light source was turned on and off as follows: 5 seconds on, 2 seconds off, then 2 minutes on, broken for 2 seconds every 30 seconds. (These are model times, so that a "3 minute surge" has an actual period of about 5.8 seconds). Therefore, the *beginning* of each reflector line has a short streak and then a break. During this test, the evening was cool, clear, and calm, with no perceptible breeze. A total of 210 reflectors were used in the basin. The results are presented in Figures 157 and 158. Figure 157 is the build-up period, commencing with the starting of the surge machine, and Figure 158 is the decay period, commencing as the surge machine was shut down. The times shown are counted from the beginning of the respective period.

It will be observed that during the build-up period the oscillatory motion attained its full amplitude in five minutes or less. The drift current increased more slowly, requiring about fifteen minutes to reach its maximum velocity. The most striking feature of the drift pattern is the large circular eddy which forms in part of the administrative area of the Naval Operating Base. This is quite well defined at the end of ten minutes.

The records taken during the decay period are rather surprising. The oscillating motion damps out very rapidly. At the end of the first 30 seconds the amplitude is not over half the steady state value, and at the end of two minutes the oscillations are hardly visible. The drift current persists much longer. At the end of five minutes the velocity is still about a quarter of the steady state value, and the large eddy is plainly visible. At the end of ten minutes the velocity is about one-eighth of the original and there is little trace of the eddy. However,



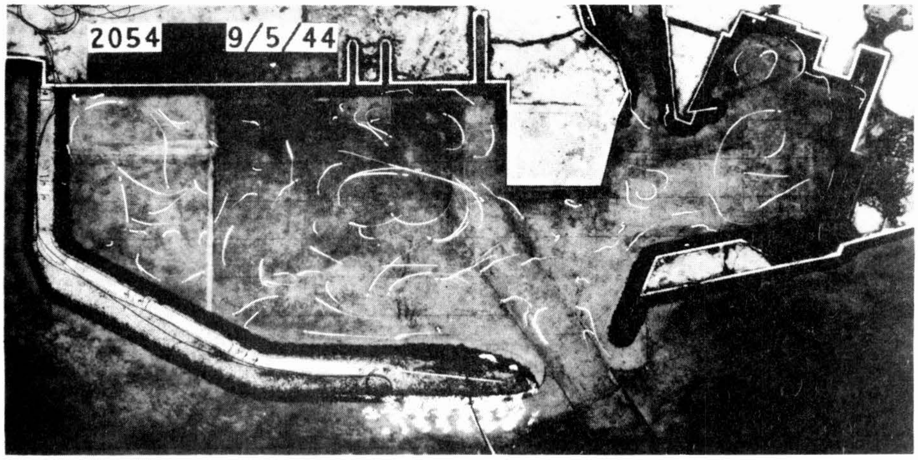
15 SEC. WAVES

3 MIN. SURGE

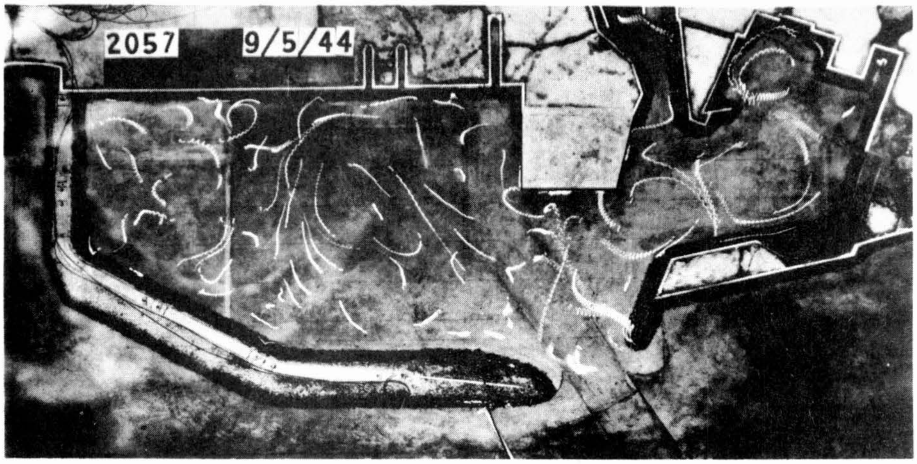
6 MIN. SURGE

FIG. 150 HORIZONTAL WATER MOTION WITH 2070 FT. GATE OPENING

15 SEC. WAVES



3 MIN. SURGE



6 MIN. SURGE

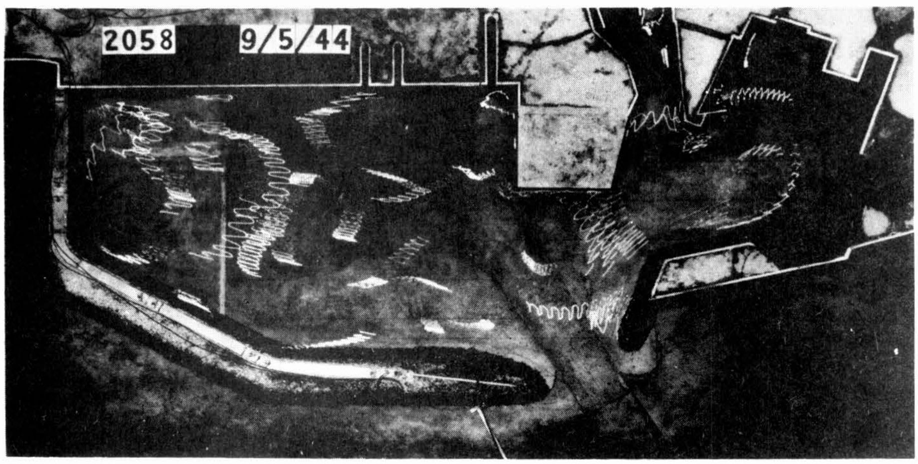
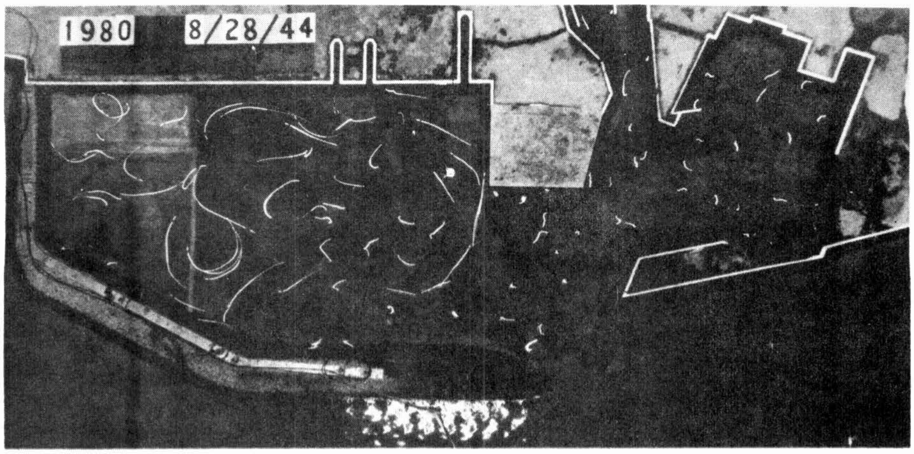
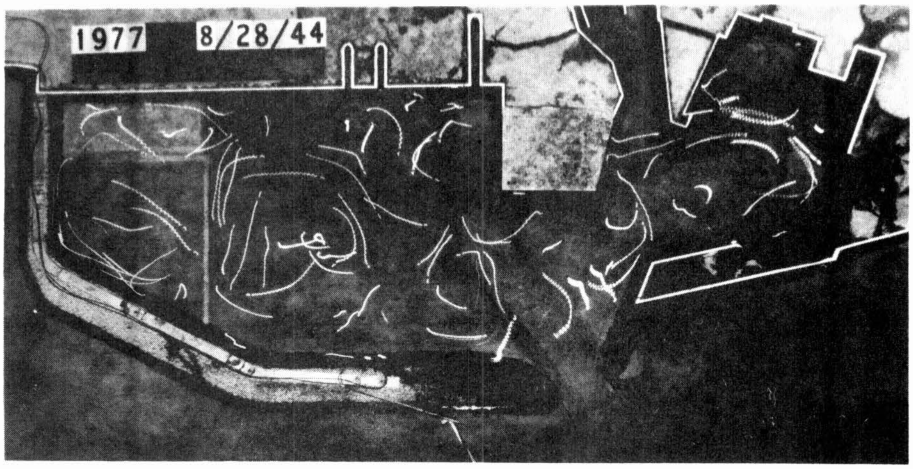


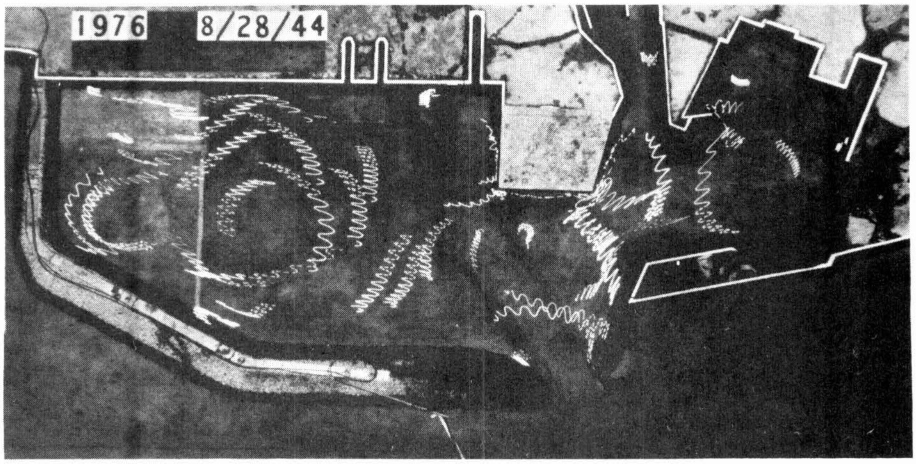
FIG. 151 HORIZONTAL WATER MOTION WITH 1320 FT. GATE OPENING



15 SEC. WAVES



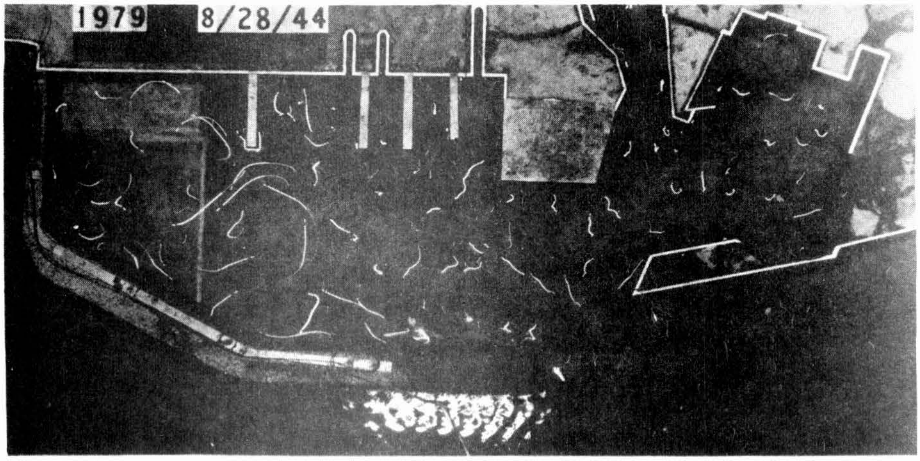
3 MIN. SURGE



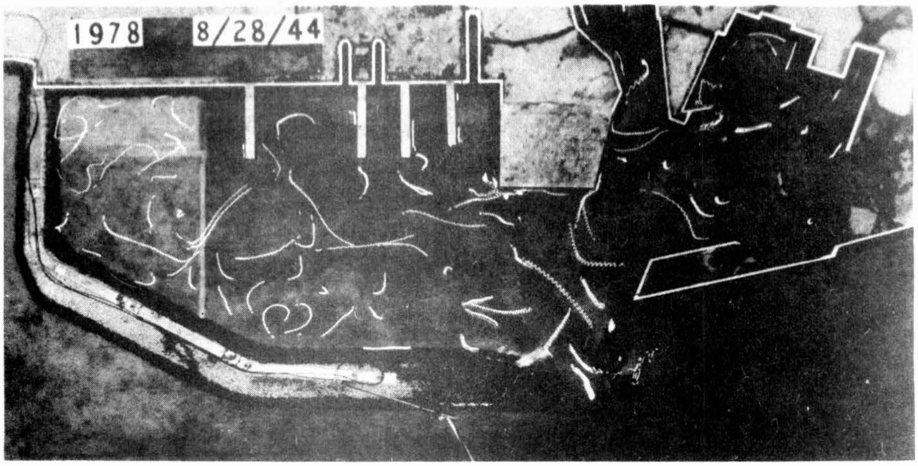
6 MIN. SURGE

FIG. 152 HORIZONTAL WATER MOTION WITH 750 FT. GATE OPENING

15 SEC. WAVES



3 MIN. SURGE



6 MIN. SURGE

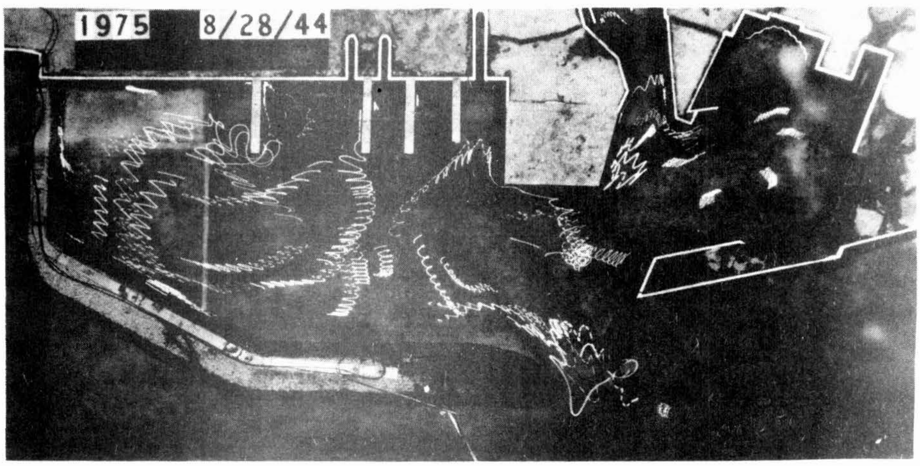
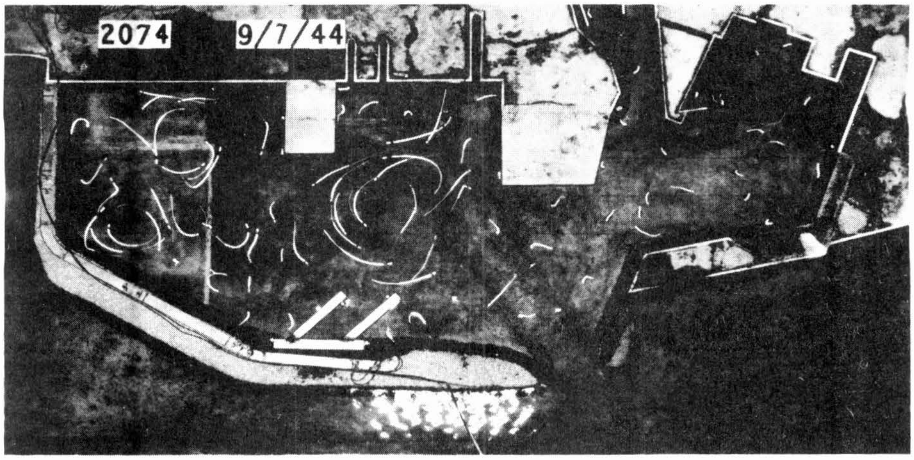
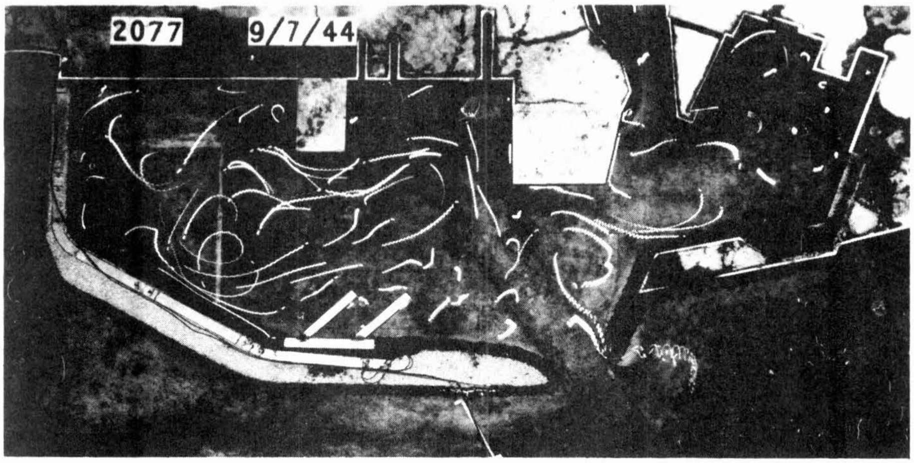


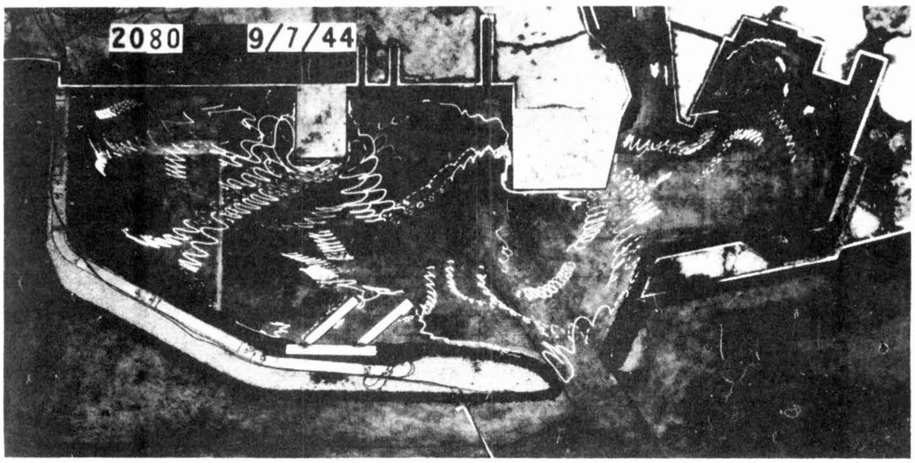
FIG. 153 MOTIONS WITH OPAQUE PIERS ADDED
750 FT. GATE OPENING



15 SEC. WAVES



3 MIN. SURGE

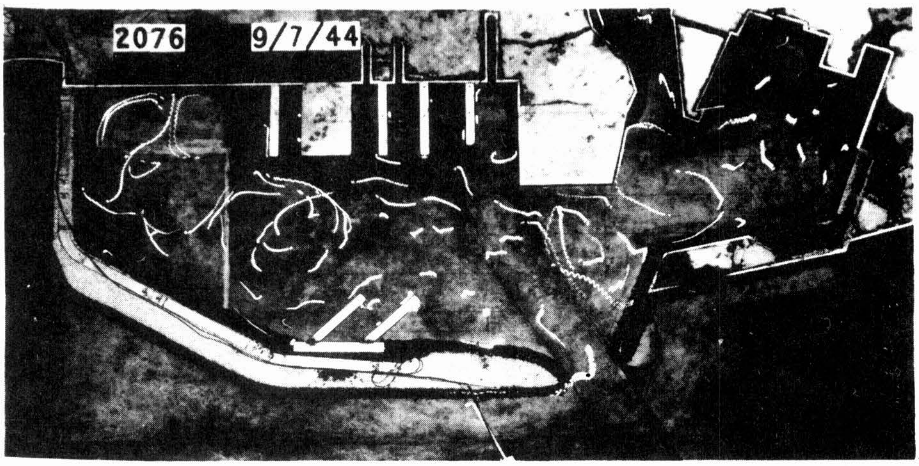


6 MIN. SURGE

FIG. 154 MOTIONS WITH DRYDOCK AND SLIP ADDED
750 FT. GATE OPENING

15 SEC. WAVES

3 MIN. SURGE



6 MIN. SURGE

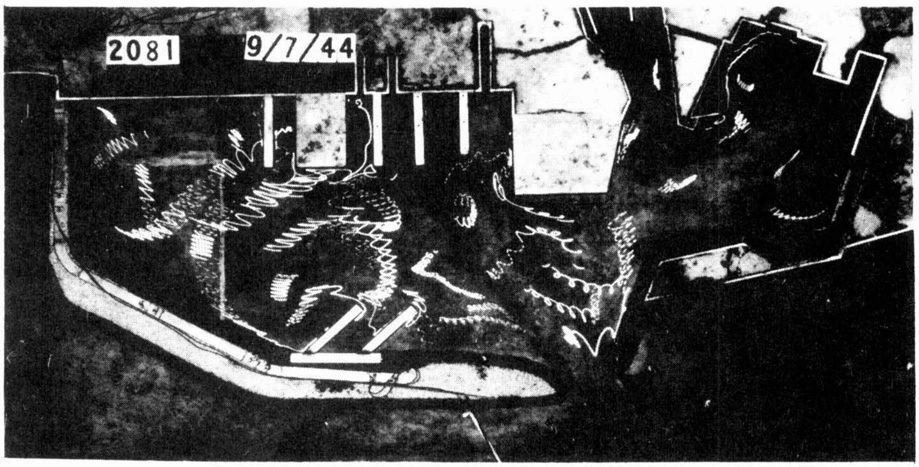


FIG. 155 MOTIONS WITH OPAQUE PIERS, DRYDOCK AND SLIP
750 FT. GATE OPENING

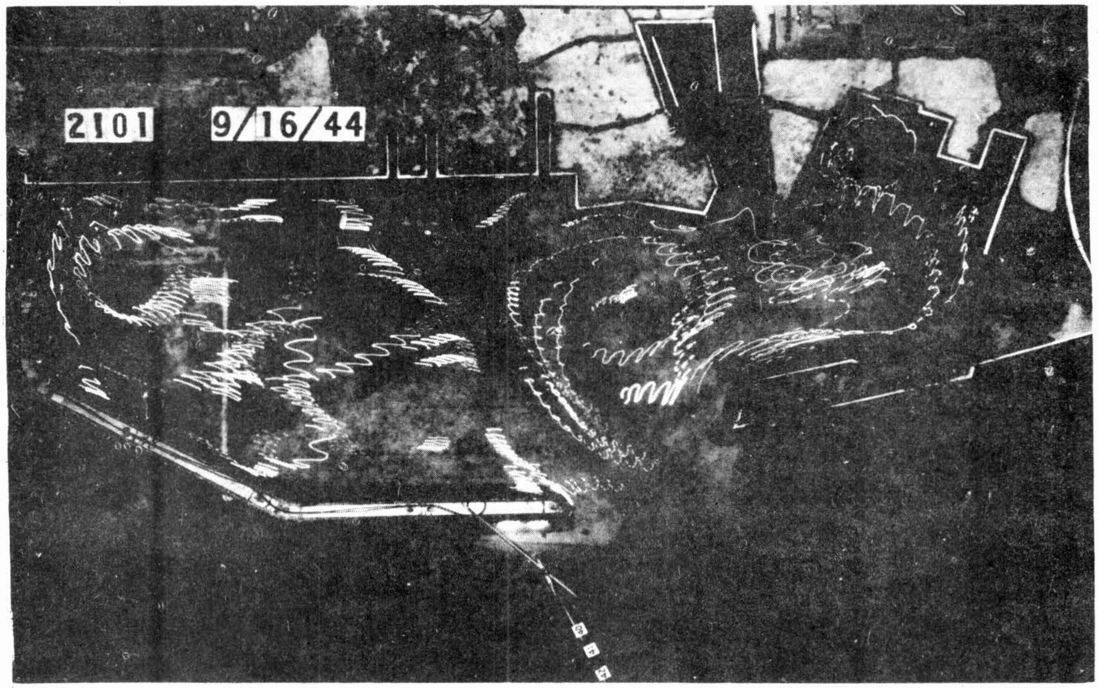


FIG. 156 MOLE BASIN WITHOUT ADDITIONAL STRUCTURES
6 MINUTE SURGE - 2070 FT. GATE OPENING

the results for fifteen, twenty and twenty-five minutes show appreciable increases in drift velocities, whereas those for thirty and thirty-five minutes indicate considerable decay. This reversal of trend is probably due to a very slight breeze, too faint to perceive and report.

The conclusions from this special study are that the oscillating motion is strongly driven and strongly damped, and the motions observed are reliable and consistent. The drift currents due to the surge machine damp out sufficiently in ten minutes to permit the start of a new run. However, the drift directions and velocity are much less reliable than are the oscillation directions and amplitudes.

(2) Horizontal velocity charts. It was felt that these reflector photographs were rather difficult to evaluate from a purely visual study. Therefore, charts were prepared showing the horizontal velocities. It was decided to indicate the direction and velocity of the drift, as well as that of the cyclic surge movement, even though it was realized that the former was not as reliable as the latter. In these maps the continuous line shows the direction of the current, or drift. The velocity of the current is given in feet per second. In many of the long lines it will be observed that there are two or more velocities indicated on one line. This means that the velocity is apparently changing along the direction of flow. The velocities are, therefore, to be interpreted as being the value corresponding to the given location in the basin. On the maps for the three minute and six

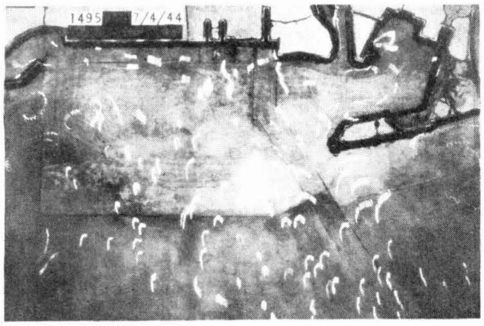
minute surge conditions the surge velocities are indicated by double-headed arrows which cross the line of the current direction at an angle. The surge arrow lies in the direction of the surge motion. It will be observed that, in general, this is not at right angles to the current, or drift. Furthermore, it will be seen that in the majority of locations the surge velocity is considerably higher than the drift velocity. In the case of the motion produced by the fifteen second wave trains, the horizontal amplitude of the cyclic motion was too small to permit the determination of the velocity. However, a careful examination of the original photographs of the reflector shots for the fifteen second waves showed that even with this high frequency motion, the cyclic oscillation was superimposed upon the drift.

(3) Comparison of basin conditions, with and without the mole.

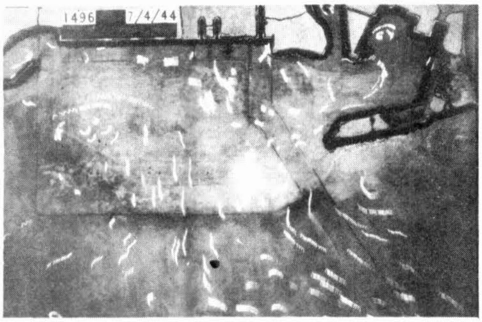
Figures 159, 160 and 161 show the motion in the basin without the mole for the fifteen second, three minute and six minute exciting trains, respectively. Figures 162, 163 and 164 show the same information for the basin with the mole in place. It will be seen that Pier F extension and Pier A wharf are now included as part of the standard configuration with the mole.

It should be noted in all cases that the only horizontal oscillations that could be found were those having the same frequency as the exciting wave train. The fact that the horizontal amplitude of motion produced by the fifteen second wave trains was too small to measure in the reflector photographs indicates that the fifteen second waves cannot produce any appreciable horizontal movements of medium and large sized ships. On the other hand, if the oscillatory motion due to the three minute surges is examined and compared for the two series of maps, it will be seen that the amplitude is greatly increased. For example, it will be observed that, in the basin without the mole, at the end of Pier 1 there is an oscillating motion whose average direction is parallel with that of the pier and whose average velocity is about 2.6 ft. per second. Thus, in a half cycle of 1.5 minutes the water moves a distance of nearly 250 ft. Fortunately, this is for a six foot surge which has a vertical amplitude many times that ever observed at the harbor. If the vertical amplitude is reduced, the horizontal amplitude should be decreased by the same ratio. This would indicate that a three minute surge with a vertical amplitude of 6 inches would result in a horizontal oscillatory motion of about 20 feet amplitude. This is very much in accordance with observations of the ship motions due to surge at the harbor.

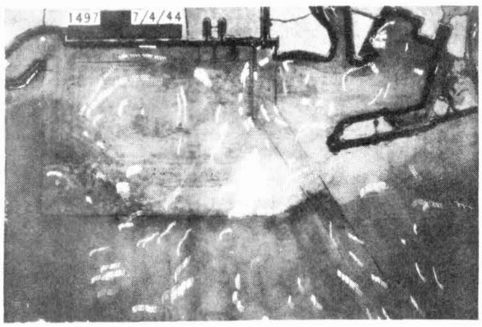
Now, if conditions caused by the six foot surge without the mole are compared to those shown to exist in the model with the mole and Pier E extension in place, it will be seen that the average surge velocity had been reduced to .2 ft. or .3 ft. per second or, in other words, by a factor of 8 to 12. This is probably a somewhat optimistic estimate of the reduction produced by the mole, since a similar comparison for the horizontal motions which are caused by an exciting surge having a period of six minutes shows that the mole has much less effect on reducing motions of



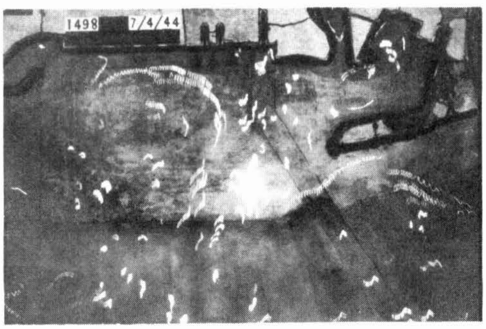
2 MINUTES



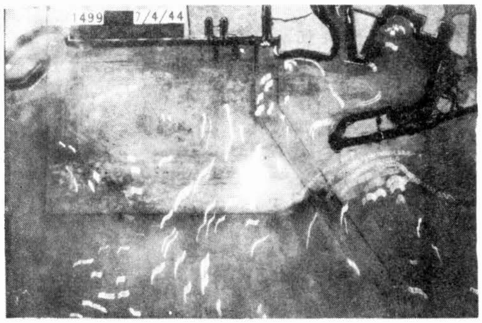
5 MINUTES



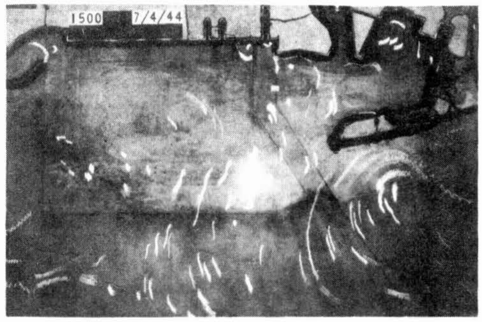
10 MINUTES



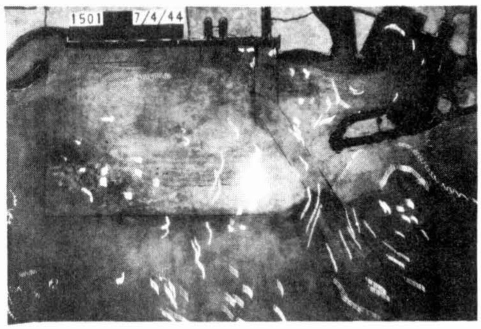
15 MINUTES



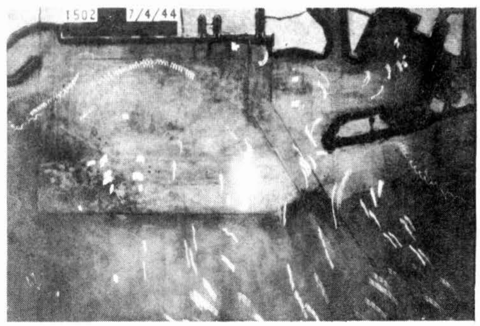
20 MINUTES



25 MINUTES

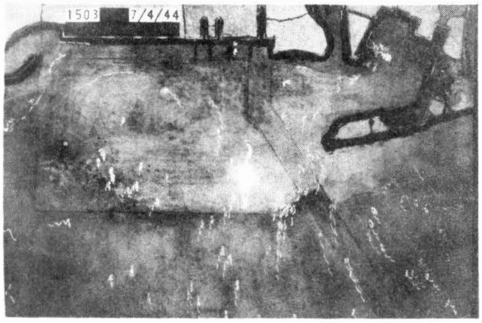


30 MINUTES

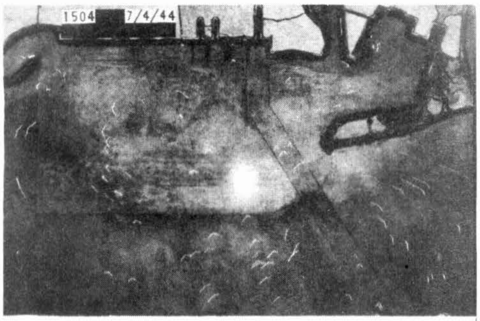


35 MINUTES

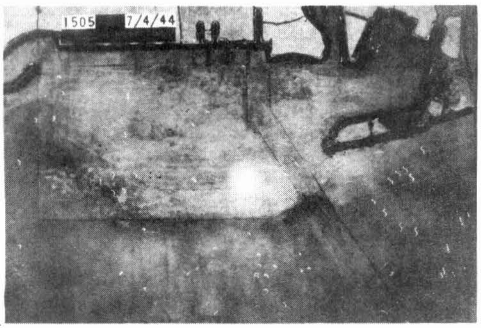
FIG. 157 SURGE OSCILLATION AND DRIFT CURRENT STUDY
BUILD UP PERIOD



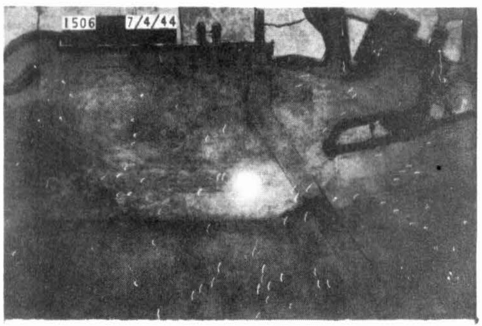
0 MINUTES



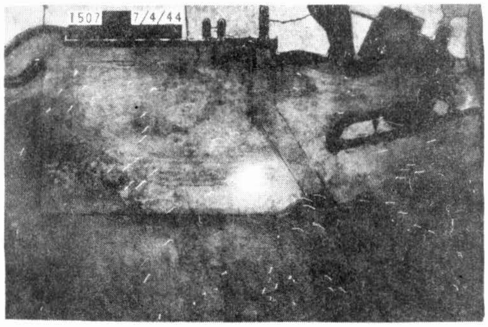
5 MINUTES



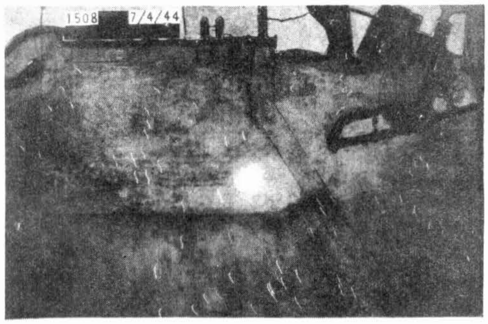
10 MINUTES



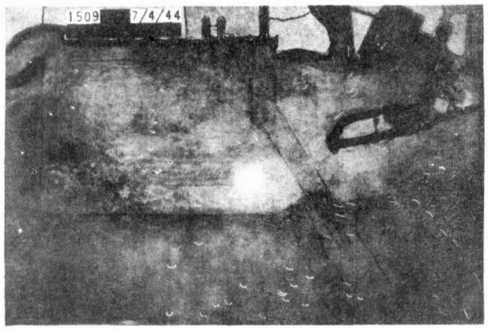
15 MINUTES



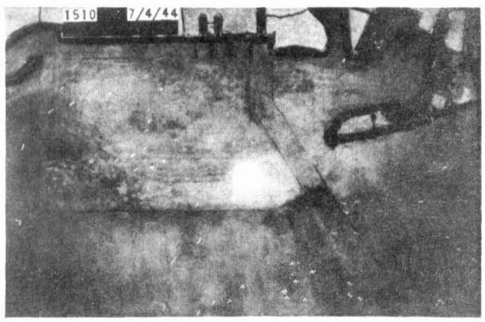
20 MINUTES



25 MINUTES



30 MINUTES



35 MINUTES

FIG. 158 SURGE OSCILLATION AND DRIFT CURRENT STUDY
DECAY PERIOD

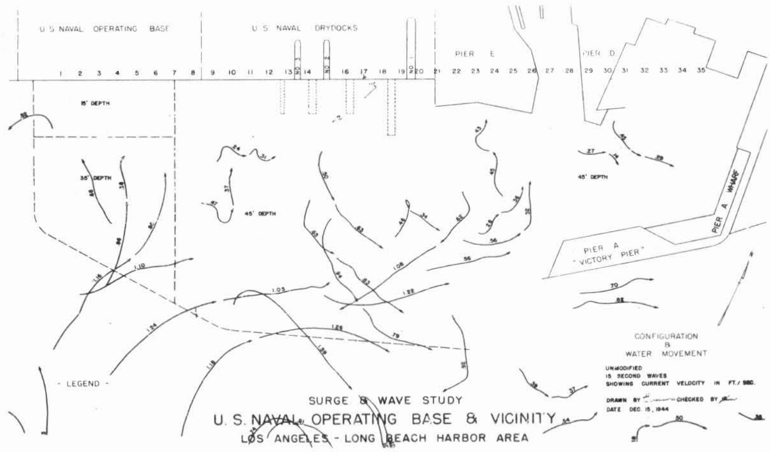


FIG. 159 HORIZONTAL MOVEMENT CHART - 15 SECOND WAVES
UNMODIFIED BASIN.

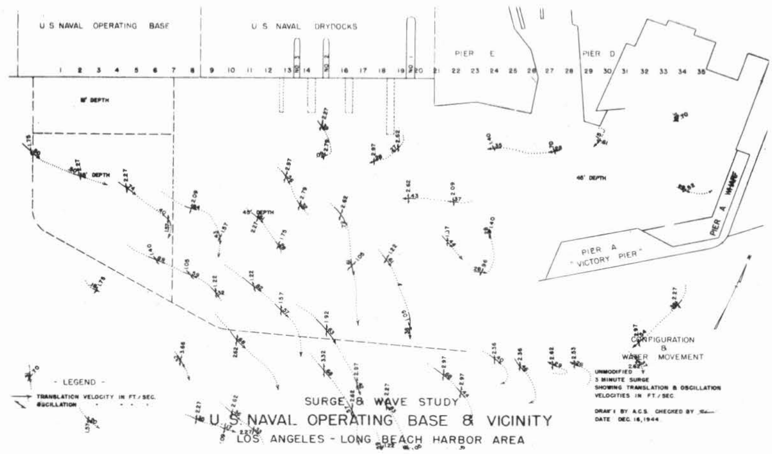


FIG. 160 HORIZONTAL MOVEMENT CHART - 3 MINUTE SURGE
UNMODIFIED BASIN

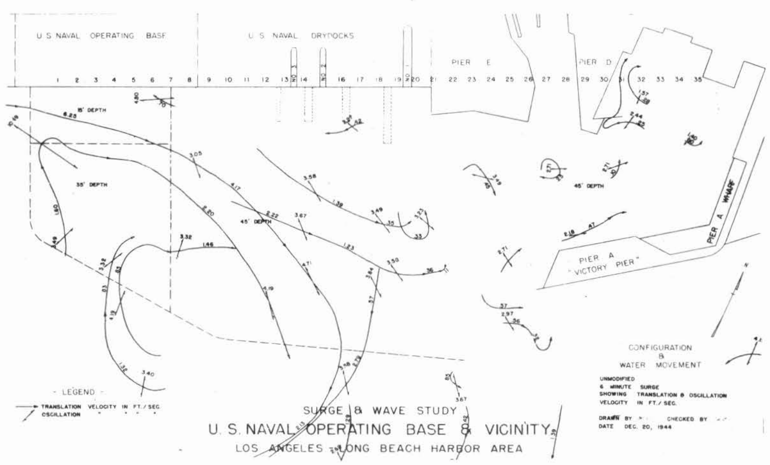


FIG. 161 HORIZONTAL MOVEMENT CHART - 6 MINUTE SURGE
UNMODIFIED BASIN

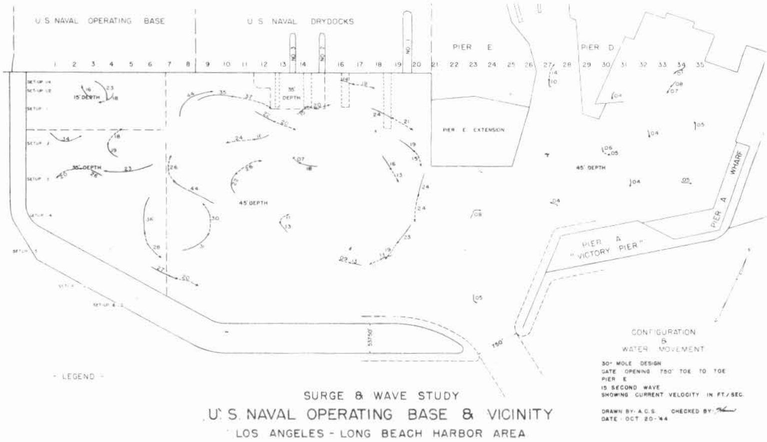


FIG. 162 HORIZONTAL MOVEMENT CHART - 15 SECOND WAVES, MOLE IN PLACE, 750 FT. GATE

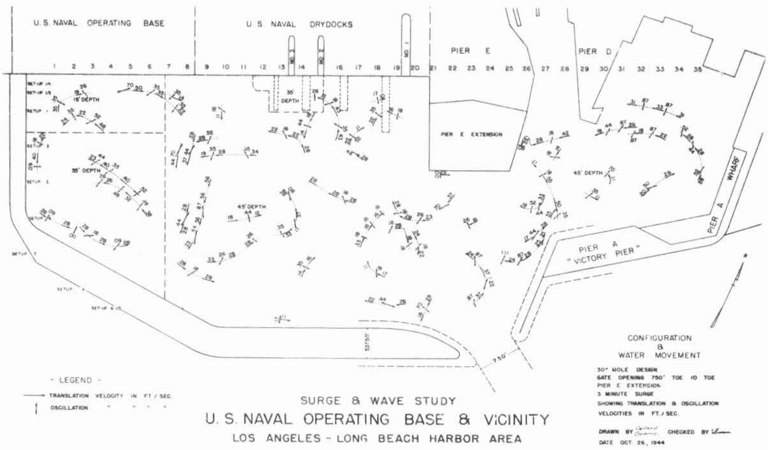


FIG. 163 HORIZONTAL MOVEMENT CHART - 3 MINUTE SURGE MOLE IN PLACE, 750 FT. GATE

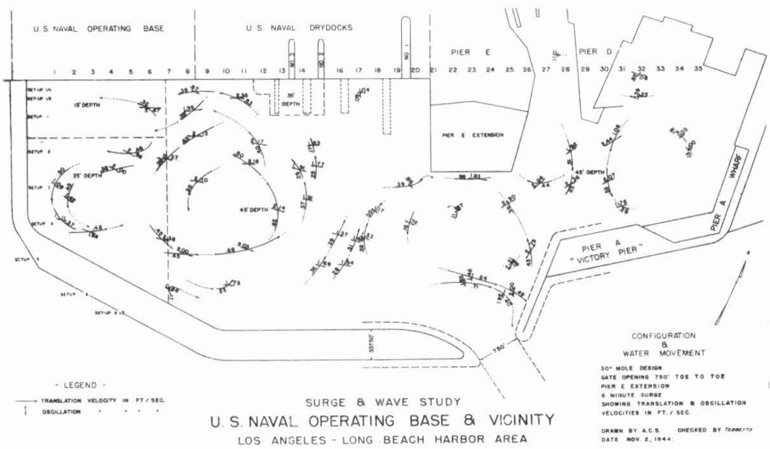


FIG. 164 HORIZONTAL MOVEMENT CHART - 15 SECOND WAVES, MOLE IN PLACE, 750 FT. GATE

this period than any other. Thus for the six minute surge, between Piers 1 and 2 without the mole in place the average velocity of the oscillating current is about $2\frac{1}{4}$ ft. per second, whereas, with the mole in place, it has been reduced to 1-1/8 ft. per second. At a point near the east limit of the area of 15 ft. depth an approximately equal ratio of reduction has taken place; from .7 ft. per second to .38 ft. per second. Near the end of Victory Pier, however, the average velocity without the mole is apparently 2.7 ft. per second and with the mole it seems to vary from 2 ft. to 2.7 ft. per second. Likewise, in the Long Beach harbor area the average velocity seems to be about the same with and without the mole.

The same comparison with and without the mole for the fifteen second waves shows that the drift velocity is considerably reduced by the presence of the mole.

Figures 161 and 164 again emphasize the very serious conditions which will be produced by six minute surges. The velocity of oscillation with these six minute surges with the mole in place, the 750 ft. gate, and Pier E extension, is as high as it is for the three minute surges without the mole at all. The fact that the half-oscillation lasts twice as long increases the seriousness of this condition. Thus, for example, in the areas in which the oscillation velocity is 2 ft. per second, a condition which is fairly general within the basin, the water would move a distance of about 360 ft. in the 180 seconds of the half-cycle, assuming the six ft. vertical amplitude of the test surge. If this amplitude were reduced to, say, 1/10 of a foot; a quantity too small to be detected by any existing tide gages, especially when overlaid by a fifteen second wave train and by surface chop, a resulting horizontal motion of as much as 6 ft. should be expected.

(4) Gate opening. The effect of variations in the width of the gate opening can be studied further by an examination of Figures 165, 166 and 167. In these figures only the conditions for the three minute surges are shown. These maps are, therefore, directly conparable to Figures 160 and 163 for conditions without the mole and with the 750 ft. gate opening, respectively. The following table shows that the amplitude of the horizontal motion reduces consistently as the gate opening is decreased. In this table certain areas have been selected for the comparison and the velocities of the oscillations are read from the map. It is impossible to obtain great accuracy in this comparison since it will be observed that the reflectors, and therefore readings, were never in exactly the same position in the basin. However, the trend is clear and unmistakable.

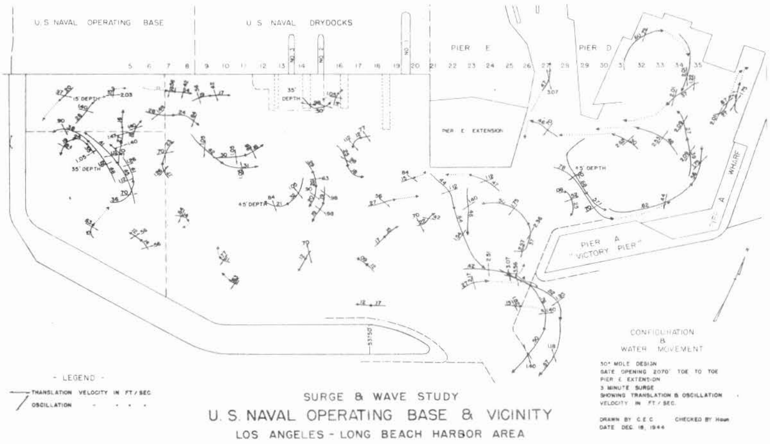


FIG. 165 HORIZONTAL MOVEMENT CHART — 3 MINUTE SURGE
2070 FT. GATE OPENING

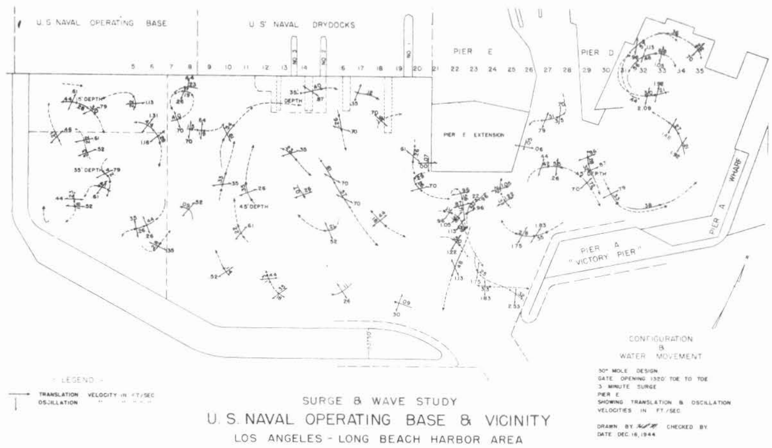


FIG. 166 HORIZONTAL MOVEMENT CHART — 3 MINUTE SURGE
1320 FT. GATE OPENING

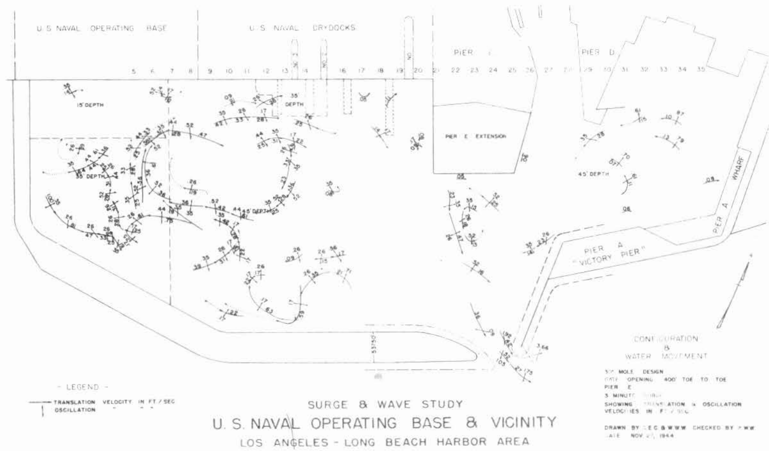


FIG. 167 HORIZONTAL MOVEMENT CHART — 3 MINUTE SURGE
400 FT. GATE OPENING

AVERAGE OSCILLATION VELOCITY FOR 3 MINUTE SURGE
FEET PER SECOND

	Pier #1	NW Corner basin	Diagonal leg of mole	Parallel leg of mole
Without mole	2.5-3.0	1.75-2.25	1.00-1.40	1.05-1.90
2070 ft. gate opening	.8-1.1	.90-1.40	.55- .60	.60- .70
1320 ft. gate opening	.35-.70	.60-1.10	.35- .52	.25- .35
750 ft. gate opening	.20-.35	.25- .50	.00- .20	.17- .44
400 ft. gate opening	.17 ±	.26- .35	.25- .35	.17 ±

<u>(5) Effect of additional structures</u>. The effects of adding structures within the basin are shown in Figures 168, 169 and 170 and Figures 171, 172 and 173. Here again, only conditions for the three minute surges are shown. It will be observed that all of the maps are for the 750 ft. gate opening with the pointed mole ends. Figures 168 and 169 show the horizontal movements without and with Pier E extension. It is necessary to observe these maps very critically, since the number of reflectors in the basin for the two maps was quite different and, therefore, the one with more lines is apt to be taken as indicating more motion. It will be observed that Pier E extension produces a substantial reduction in the oscillation velocity in the entire basin. This is particularly noticeable along the drydocks frontage. The reduction is not so striking in the Long Beach harbor area, or along the parallel leg of the mole. The drift pattern will be seen to be essentially unchanged. In the balance of the maps it will be observed that Pier E extension remains in place. Figure 170 shows the effect of adding the drydocks to the basin. This addition has little apparent effect. Figure 171 presents the conditions with Piers 1 to 4 opaque. One of the obvious results is that there is no longer any detectable motion normal to the piers. However, the drift motion parallel to the pier is reduced to a low value. Apparently the addition of the opaque piers also tends to quiet the motion in the remainder of the basin. Figure 172 is a map for the basin with Pier E extension and the mole piers and marginal wharf on the parallel leg. If this is compared to Figure 169, which is for the same configuration but without the mole piers and marginal wharf, it will be seen that the main results of the installation of the mole piers is to quiet conditions along the diagonal leg. Movement within the mole slip is not detectable. There apparently are rather severe conditions existing at the ends of these piers. In Figure 173 it will be seen that the drydock has been added with about the results that would have been expected from a consideration of Figures 170 and 172. Figures 174 and 175 have been constructed to show an over-all comparison between the empty basin and the basin in the fully developed condition. It will be observed that in Figure 175 all of the structures, including the opaque piers, are present in the basin. This comparison shows clearly the damping effect produced by the addition of this series of opaque structures. It should be borne in mind that this damping is probably due both to the additional energy dissipation that takes place and to the multitude of reflections of the wave trains from all of these new opaque surfaces.

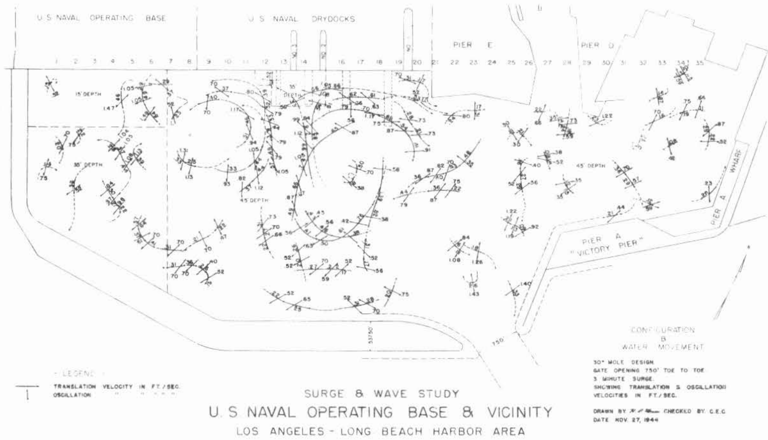


FIG. 168. HORIZONTAL MOVEMENT CHART – 3 MINUTE SURGES – 750 FT. GATE
NO ADDITIONAL STRUCTURES

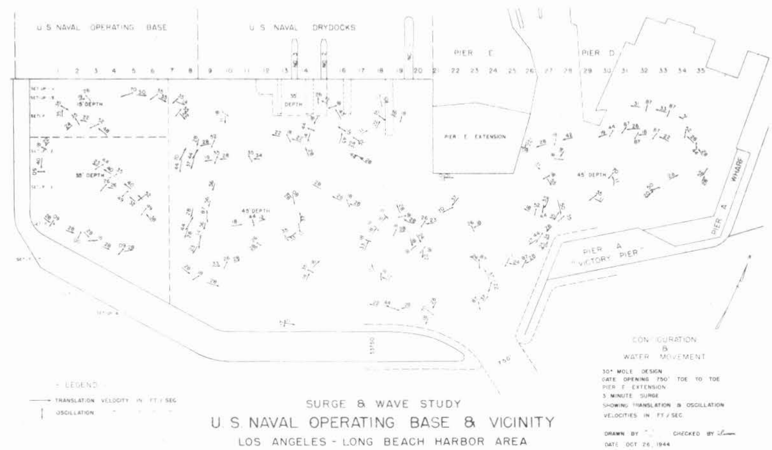


FIG. 169 HORIZONTAL MOVEMENT CHART – 3 MINUTE SURGES –750 FT. GATE
PIER E EXTENSION

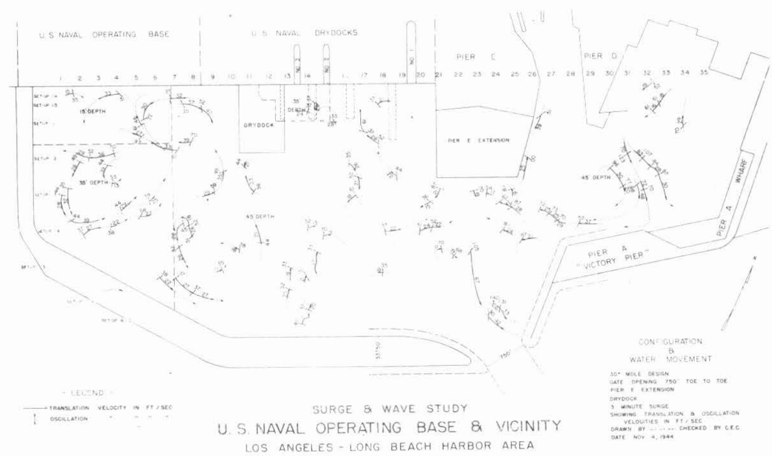


FIG. 170 HORIZONTAL MOVEMENT CHART – 3 MINUTE SURGES – 750 FT. GATE
PIER E EXTENSION AND DRYDOCKS

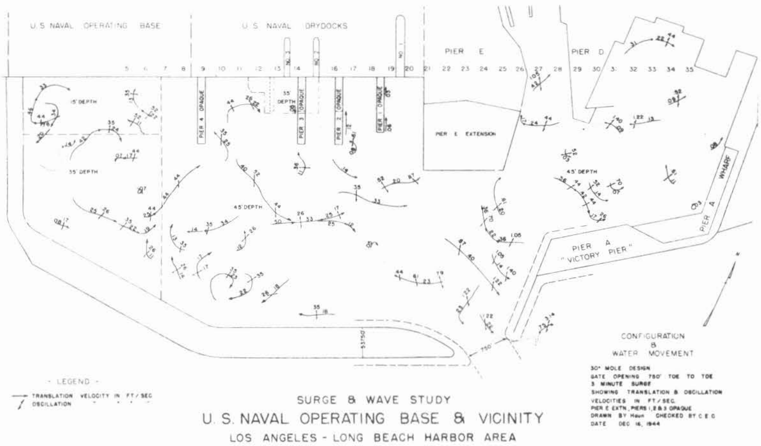


FIG. 171 HORIZONTAL MOVEMENT CHART – 3 MINUTE SURGES – 750 FT. GATE
PIER E EXTENSION OPAQUE PIERS

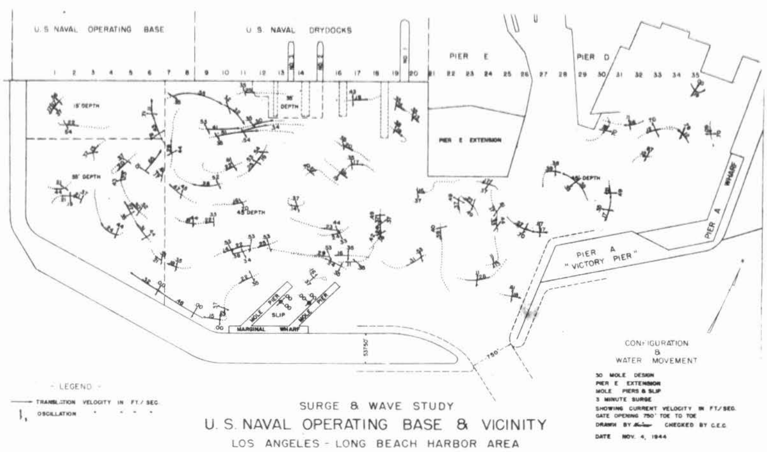


FIG. 172 HORIZONTAL MOVEMENT CHART – 3 MINUTE SURGES – 750 FT. GATE
PIER E EXTENSION AND MARGINAL WHARF WITH MOLE PIERS

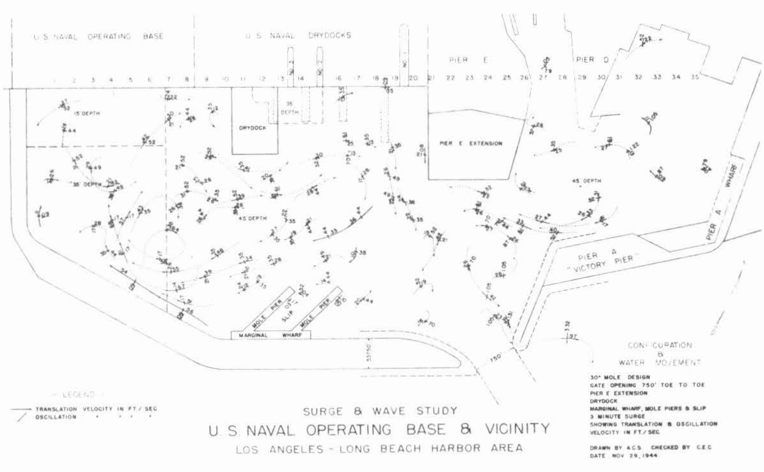


FIG. 173 HORIZONTAL MOVEMENT CHART – 3 MINUTE SURGES – 750 FT. GATE
PIER E EXTENSION, DRYDOCK AND MARGINAL WHARF WITH MOLE PIERS

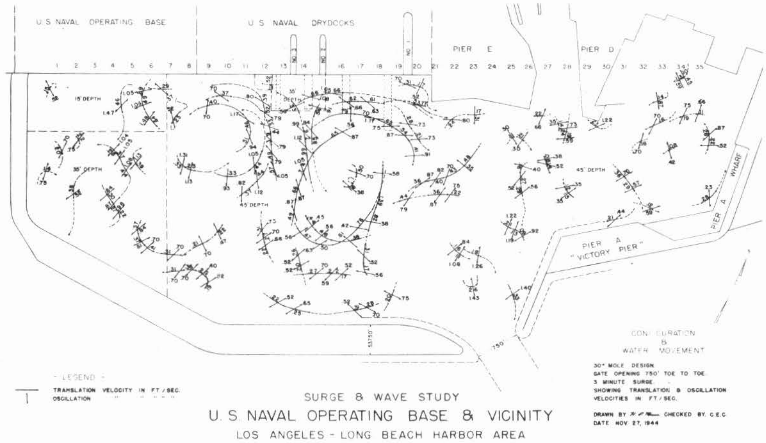


FIG. 174 HORIZONTAL MOVEMENT CHART – 4 MINUTE SURGES – 750 FT. GATE
NO ADDITIONAL STRUCTURES

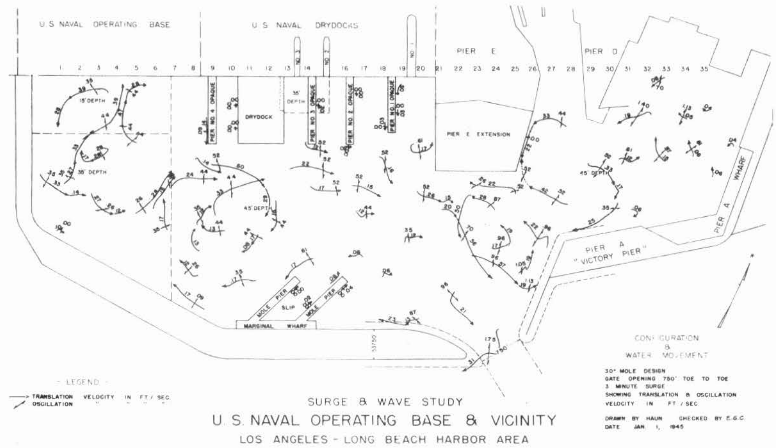


FIG. 175 HORIZONTAL MOVEMENT CHART – 3 MINUTE SURGES – 750 FT. GATE
PIER E EXTENSION, DRYDOCK, MARGINAL WHARF
WITH MOLE PIERS, AND OPAQUE PIERS

(6) General results of horizontal motion studies. A comparison of the results of these studies of the horizontal motions within the basin with the analogous studies of the vertical motions in the same area shows very clearly that results from both studies are in agreement. Thus, both studies show conclusively that the presence of the mole reduces the motion within the basin for all periods except that of the six minute surge. Both studies likewise show that conditions within the basin continually improve as the gate opening is reduced, but that below the 750 ft. width the amount of improvement that can be obtained does not seem to justify the additional inconvenience to navigation. Both studies also show that, in general, the more the shoreline is broken up by additional structures, such as opaque piers, slips, drydocks, etc., the greater the damping within the basin and the less the water movement for the same degree of excitation from the outer harbor. However, in addition to confirming the results of the studies of the vertical movement, the horizontal motion studies demonstrate very clearly the reason that the long period surges cause so much more trouble in the basin than do the relatively short period waves. Thus, it was seen that a three minute surge having an amplitude of only about 6 inches could produce a horizontal oscillating motion having an amplitude of about 20 ft., whereas, a fifteen second wave of the same amplitude would result in a horizontal motion of only one or two feet.

The difficulty of predicting the horizontal motion at any given point in the basin from the known vertical motion at that point, or even from a knowledge of the entire vertical pattern of motion for the basin, was pointed out at the beginning of this presentation. In order to illustrate this point more clearly the map shown in Figure 176 was prepared. On this map the contours of constant vertical motion are shown in dotted lines and the arrows show the horizontal motion. A condition of rather high motion was chosen to make the comparison as clear as possible. Therefore, a 2070 ft. gate opening was used with the three minute surge and with nothing in the basin except Pier A wharf and Pier E extension. It will be seen that, in general, large horizontal motions occur in areas where the vertical motion is comparatively small. There is a tendency for the horizontal motion to be approximately normal to the lines of constant vertical motion. However, there are many exceptions to these general statements. These exceptions are due primarily to the extreme complexity of the wave pattern existing in the basin.

(g) Ship movements. In Section III of this report, one of the objectives outlined for the final model studies was the study of the movement of model ships at the piers and the correlation of this movement with the vertical and horizontal water movements observed in these locations. One way of describing a model study is that the model is an integrating machine. Into this machine are put all of the individual factors that are thought to affect the performance of the area being studied. The resulting behaviour of the model is due to the inter-action or integration of all these factors. Now, one of the primary objectives of this model study was to ascertain how much reduction could be obtained in the motion of ships berthed at the drydocks and piers. Fron the studies of the horizontal and vertical motions of the water within the basin, the motions of these ships can be inferred. However, it was felt that much could be learned from an observation of the motion of model ships themselves. The results from such a study should be as valid as any other model results, since there is no more difficulty involved in modeling a ship than in modeling the basin and the wave trains that excite it.

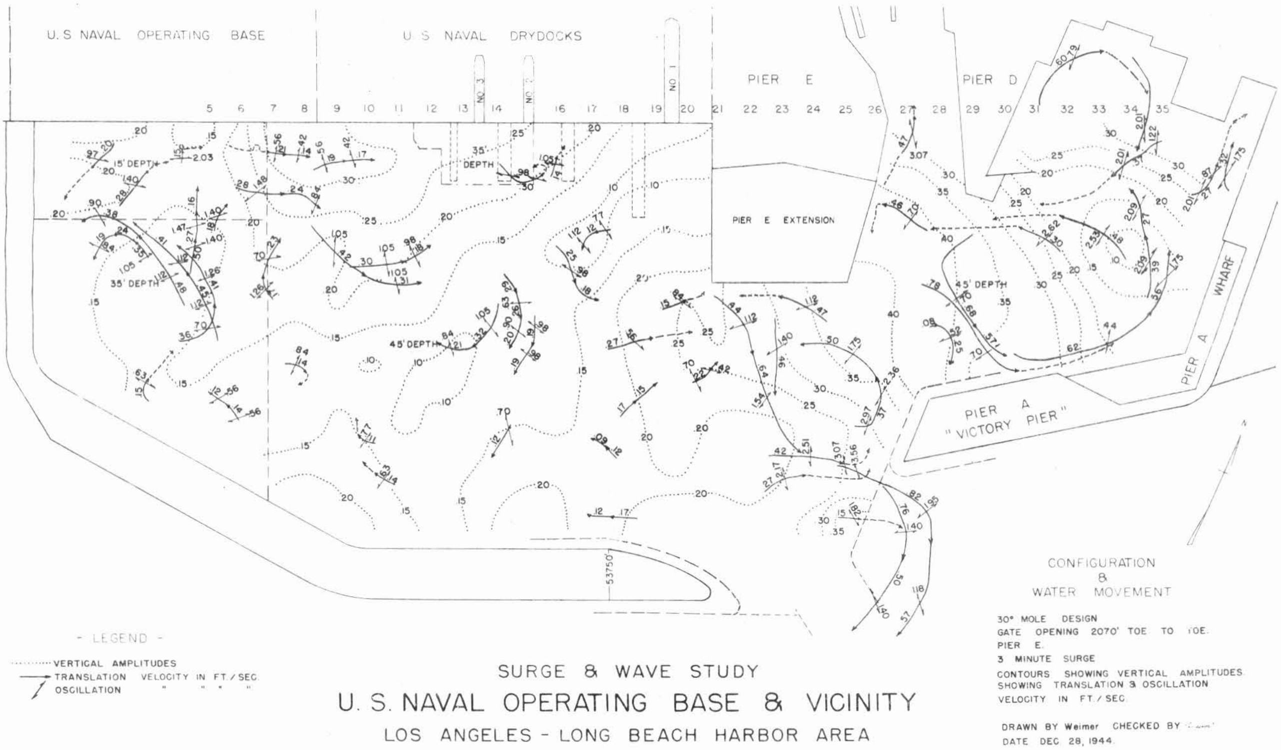


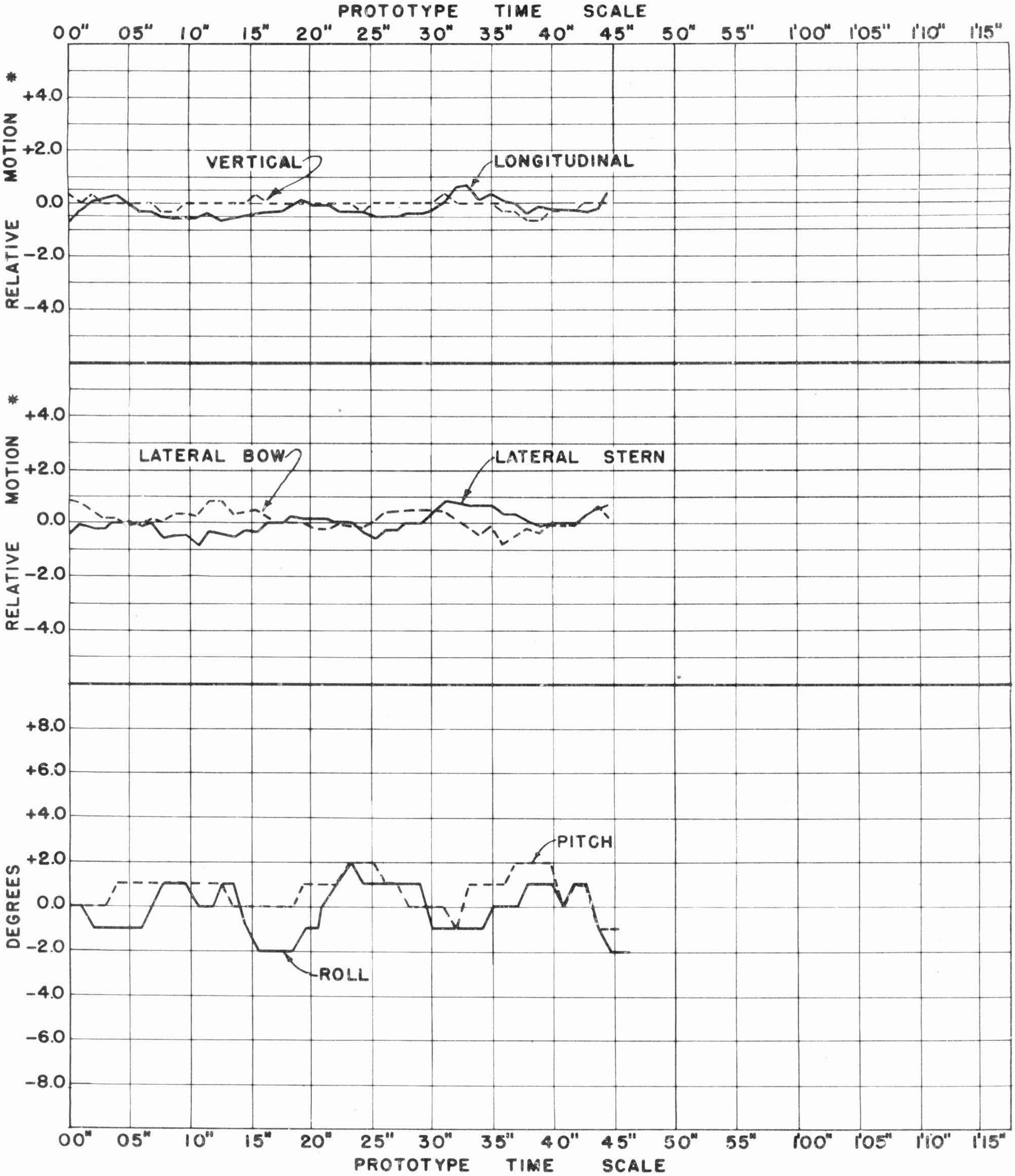
FIG. 176 VERTICAL AND HORIZONTAL MOVEMENT CAUSED BY 3 MINUTE SURGES - 2070 FT. GATE OPENING

(1) Qualitative measurements. A series of model ships was constructed from drawings furnished by the Naval Operating Base which gave the principal dimensions and typical under water contours. Information was secured also concerning the pitch and roll periods of these ships. The models were ballasted until they had the proper waterline and the correct roll and pitch periods according to the harbor model time scale. However, the model ships were made using the horizontal scale of the model basin for both the horizontal and vertical dimensions. These ships were then tied to the piers and their motions observed visually. After a period of such visual observations, a motion picture study was made of the various conditions covered by the preceding studies of the vertical and horizontal motions within the basin. The ships were fastened to the piers by loose lines which permitted a radius of motion of 40 ft. or 50 ft. A study of these pictures showed that the ship motion was very similar to the combined water motion in that area and that all factors that reduced the water motion reduced the ship motion proportionately.

(2) Quantitative measurements. In order to get a more quantitative measurement of the ship motions, a second motion picture study was made. This time the study was confined to one ship model which was assumed to be typical of many that would use the facilities of the Naval Operating Base. The type selected was a tanker, which has a length of 500 ft. and an extreme beam of 57 ft. It was berthed on the west side of Pier 1. A series of scales was fastened to the pier and the ship was provided with bow and stern pointers, a pointer amidships and a pointer on the mast. A set of mirrors was installed so that one motion picture camera installed vertically above the model and pointing downward was able to record all of the ship motions, i.e., the horizontal motions of the bow and the stern both longitudinal and lateral, the roll, the pitch and the heave. Figure 57 shows a photograph of this general set-up. Motions were measured for three types of ties, i.e., no ties, loose ties, and restricted ties. The loose ties were the same as those used in the preliminary study and gave the ship a radius of motion of about 40 ft. The restricted ties reduced this radius to approximately 10 ft. and thus corresponded quite closely with normal prototype conditions. Measurements were made for fifteen second, three minute and six minute waves and surges. Approximately the same set of basin configurations were investigated for their effect on the ship movements as had been studied for the horizontal and vertical water motions. However, the study of the effect of additional structures within the basin was restricted to the comparison of the behaviour of the ships with Piers 1, 2, 3 and 4 transparent and opaque. 2070 ft. 750 ft., and 600 ft. gate openings were studied. In all of the runs with the mole in place, Pier A wharf and Pier E extension were in the basin. Runs were also taken without either the mole or Pier E extension.

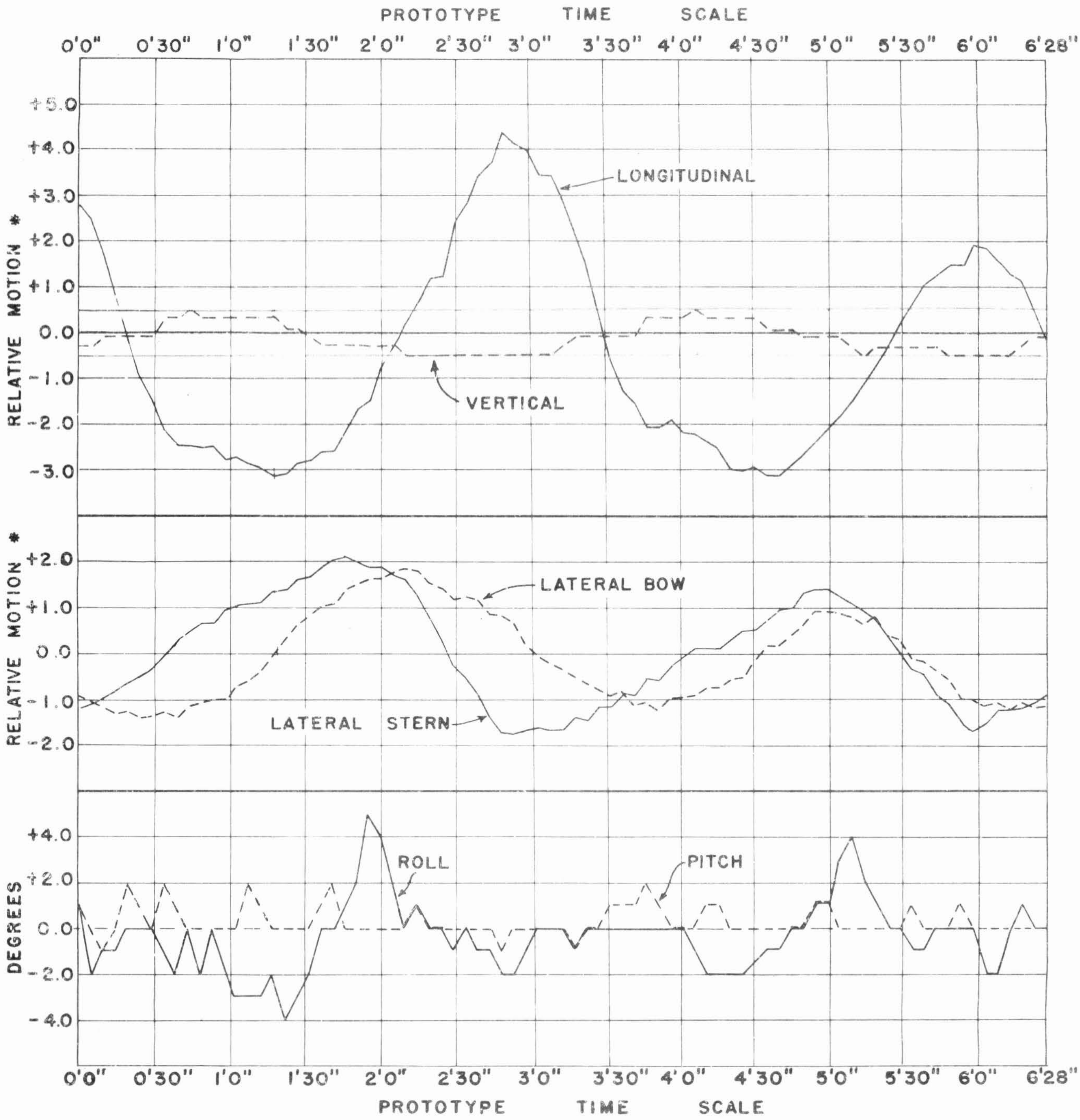
To evaluate the ship motion the individual frames of the motion picture film were projected and the readings of the pointers were tabulated. These readings were then corrected to eliminate the effect of the length of the pointers and also the optical distortion through the mirror system. Thus, the results as presented here represent the actual ship movements expressed in prototype dimensions. An examination of the results showed that the runs with no ties had little significance because the drift was enough to move the ships so that the pointers moved off the scales before a cycle could be duplicated. Therefore, the only measurements that were evaluated for the no-tie runs were for the three minute surge with no mole and for the 750 ft. gate with and without opaque piers. The experiments with loose ties and the restricted ties were very satisfactory. The results of these runs furnish the basis for the following discussion.

(3) Comparison of ship motion caused by the three wave trains without the mole. One of the first items to be investigated in the study of ship motion is the relative effect of wave trains of different periods on the motions of the ship. This effect is seen very clearly by comparing the motion of the ship without the mole when subjected successively to three different wave trains. Figures 177, 178 and 179 show the measured ship motions due to



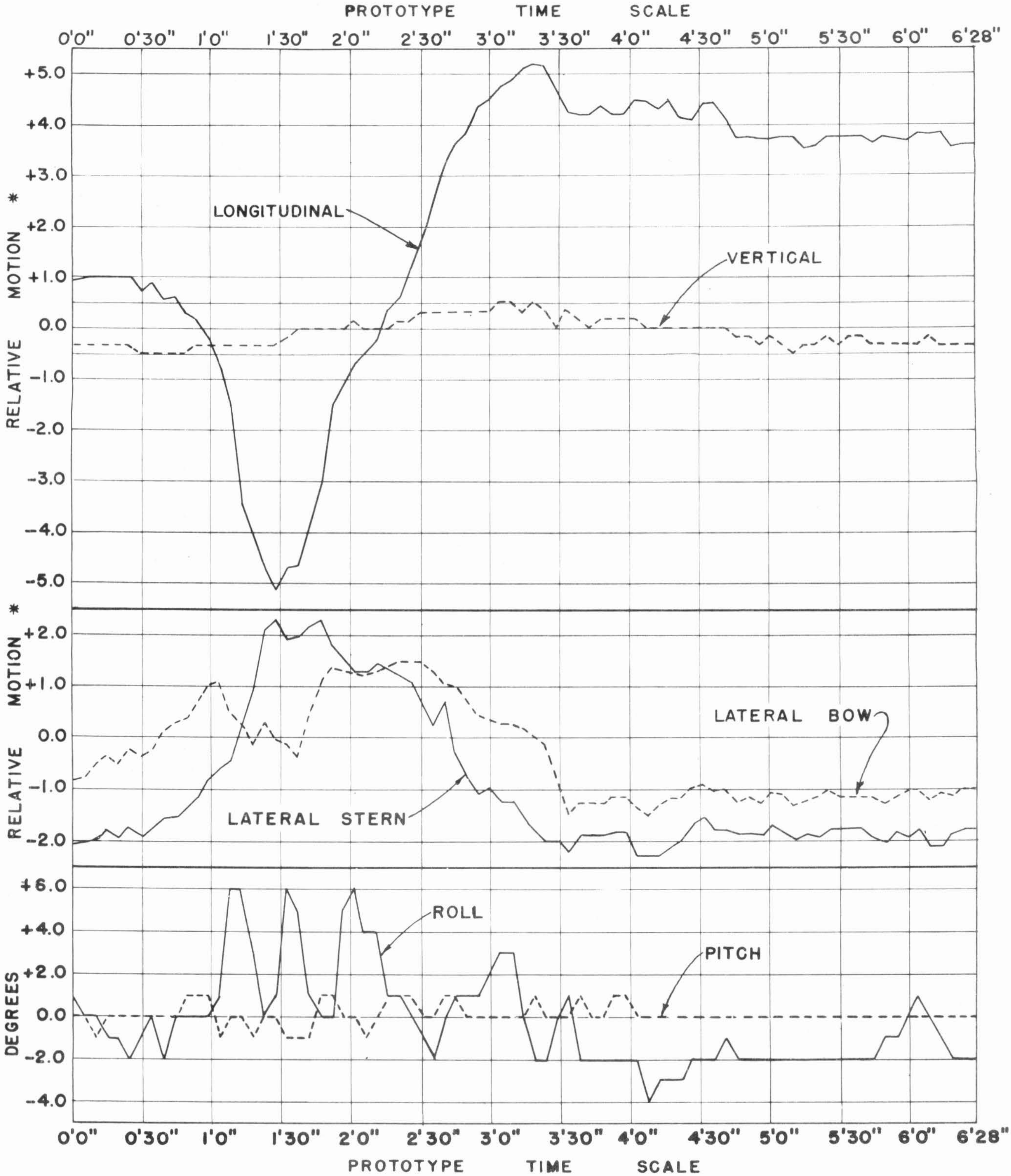
* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 15 SECOND WAVES

FIG. 177 SHIP MOTIONS CAUSED BY 15 SECOND WAVES WITHOUT MOLE IN PLACE - LOOSE TIES



* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 3 MINUTE SURGE

FIG. 178 SHIP MOTIONS CAUSED BY 3 MINUTE SURGE WITHOUT MOLE IN PLACE - LOOSE TIES



* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 6 MINUTE SURGE

FIG. 179 SHIP MOTIONS CAUSED BY 6 MINUTE SURGE WITHOUT MOLE IN PLACE – LOOSE TIES the fifteen second wave train, the three minute surge train, and the six minute surge train, respectively. In all cases the loose ties were used. In examining these diagrams it should be noted that the unit of motion used is the vertical amplitude of the wave trains causing the ship movement. In other words, the figures on each chart for the four linear motions, i.e., the longitudinal movement along the pier, the lateral movement of the bow, the lateral movement of the stern, and the vertical movement of the center of gravity, can all be thought of as indicating the feet of motion per foot of wave height. The angular motion of pitch and roll are simply expressed in degrees. The abscissa for all diagrams is the prototype time scale. An examination of the three diagrams shows that in all cases the periods of the three horizontal components of the linear motion are the same as that of the exciting wave train. For example, on the diagram for the fifteen second wave train, it is seen that maxima of longitudinal motion occur at four seconds, nineteen seconds, and thirty-three and one-half seconds, i.e., at an interval of about fifteen seconds. The lateral bow motion shows a minimum at about twelve seconds and another at twenty and one-half seconds. The lateral stern motion has peaks at five seconds, twenty seconds, and about thirty-four seconds. The same cyclic behavior is also evident on the diagram for the three minute surges and the trend is also visible on that for the six minute surges, although in the latter case the abscissa is too short to trace more than one complete cycle. Thus, it is evident that these motions are forced vibrations for the ship. Furthermore, in the first series of motion pictures simultaneous trains of fifteen second waves and three minute surges were introduced in the basin. It was clearly observable that the ship had a compound motion made up of both of these periods. One important corollary to this conclusion that the ship oscillates at the same frequency as that of the exciting wave trains is that the horizontal motions of the ships at the piers in the harbor furnish definite indications of the periods of the various wave trains existing in the area at that time. The vertical motion appears to oscillate at apparently constant frequency for all wave trains. The period corresponds roughly to that of the fifteen second waves, but this is also about the natural period of the ship.

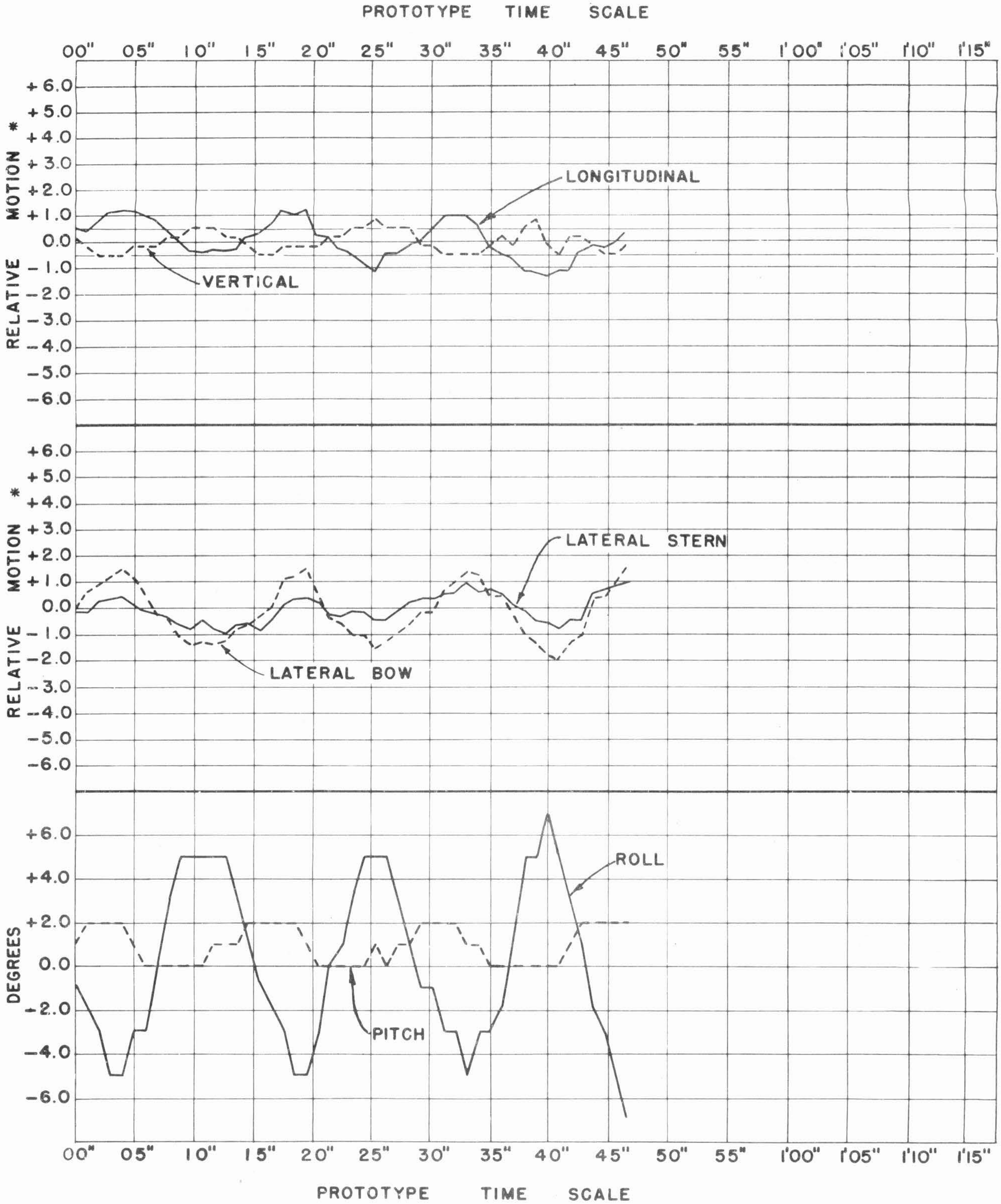
A second item which is shown by these three diagrams is that the vertical motion of the center of gravity of the ship is the same as the vertical amplitude of the exciting wave train. Thus, in all three diagrams, it will be seen that the ship oscillates between +.5 and -.5 i.e., an amplitude of one wave or surge height. This does not mean that no part of the ship has a vertical motion greater than that of the wave, but simply that the center of gravity moves that amount. The bow and stern will move through a larger distance if the ship is pitching and roll can increase the vertical motion of points not on the longitudinal centerline.

If the three remaining components of linear motion are now examined, it will be seen that their behavior differs markedly with different wave lengths of the disturbing wave train. Thus, with the fifteen second waves, it will be observed that the amplitudes of the longitudinal motion, the lateral bow, and the lateral stern motions are all about the same as that of the vertical motion, in other words, the same amplitude as the height of the wave. Thus, for example, a fifteen second wave train having an amplitude of about 2 ft. could be expected to move a ship about 2 ft. in any direction horizontally or vertically. Now, if the results for the three minute surge train are examined, it will be seen that the longitudinal motion is about seven times that of the vertical, whereas the lateral bow and stern motions are about three to three and one-half times that of the vertical motion. On the measurements for the six minute surge it will be seen that these three components are even larger. The longitudinal motion is more than ten times that of the vertical, the stern motion is nearly five times, and the bow motion is nearly three times that of the vertical. In the case of the six minute surge, it is very probable that these motions are so great that the loose ties are limiting the motion.

If these horizontal ship motions are compared with the results of the horizontal water motion studies for Model 3, it will be seen that the results parallel closely. However, there is obviously a damping factor which acts to reduce the ship motion below that of the water. Thus, a three minute surge showed a horizontal motion of some forty times that of the vertical motion, whereas the corresponding ratio for the ship motion is only about seven. This reduction in motion of the ship over that of the water can be expected, since the accelerating and decelerating forces on the ship are obtained by the action of the water flowing either faster than the ship is moving or in the opposite direction to its motion. However, it should be emphasized that the horizontal ship motion follows the horizontal water motion, and that the large horizontal water motions resulting from comparatively small vertical amplitude, long period surges are the cause of the excessive ship motion observed at the piers and drydocks.

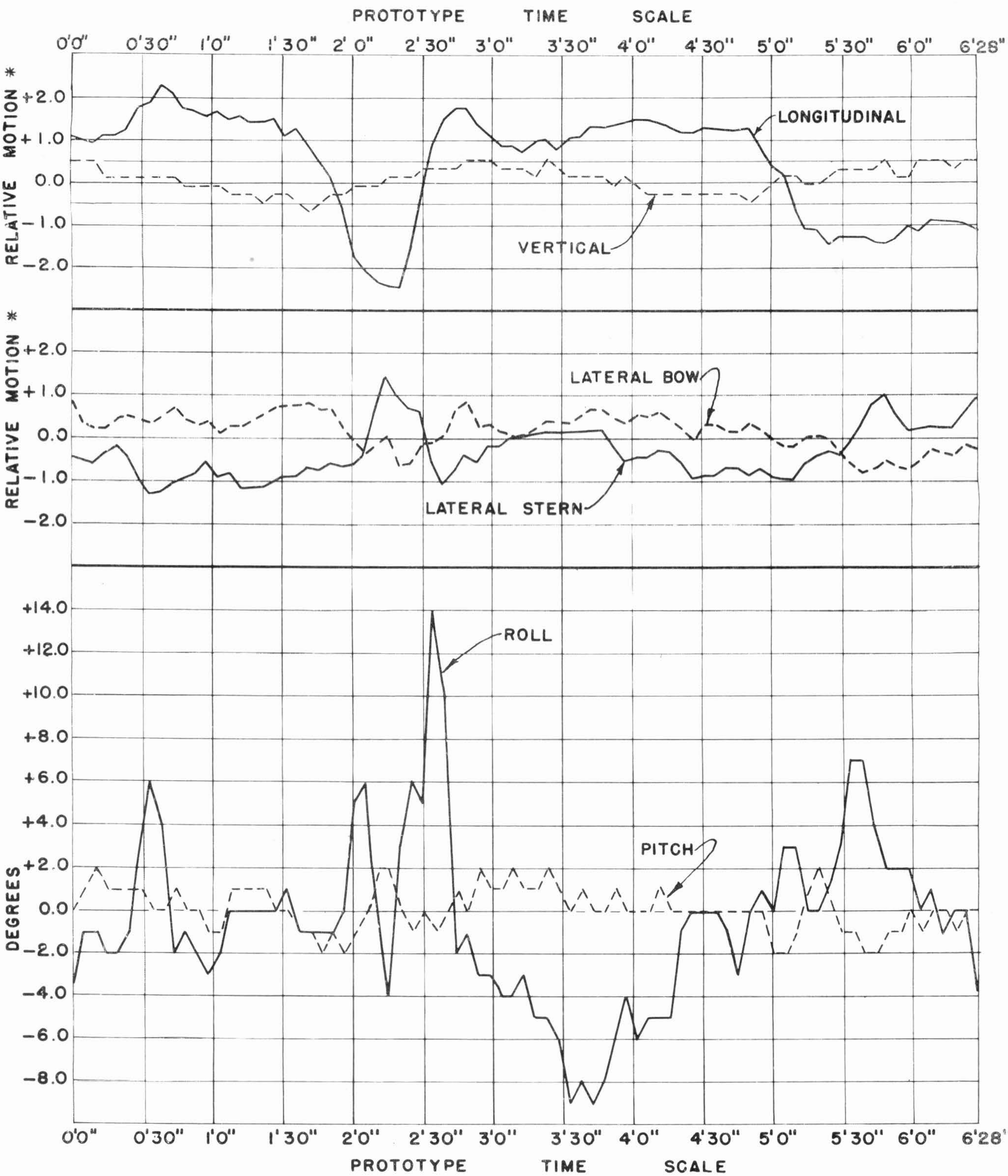
The pitch and roll are the other two components of motion observed in these studies. The pitch periods show much less correlation with the period of the wave trains than does any other component of motion. Since the ship is very "stiff" in pitch, this probably indicates that the resulting motion in pitch is a combination due to the free period of the ship and the period of the exciting train. On the other hand, the roll periods show quite uniformly a good correlation with the period of the exciting waves. In the case of the six minute train, this is complicated by the fact that the motion in roll was apparently affected by the ties. The amplitude of pitch seems to be quite independent of the period of the waves, since in all cases it seems to be about three degrees. The amplitude of roll however, increases with the length of the period, although not clearly as rapidly as do the other motions. Thus, for the fifteen second waves, the amplitude is about four degrees, for the three minute surge it is about nine degrees and for the six minute surge about ten degrees.

Figures 180 and 181 show the results of similar measurements for the fifteen second waves and the three minute surges, but with



* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 15 SECOND WAVES

FIG. 180 SHIP MOTIONS CAUSED BY 15 SECOND WAVES WITHOUT MOLE IN PLACE – RESTRICTED TIES

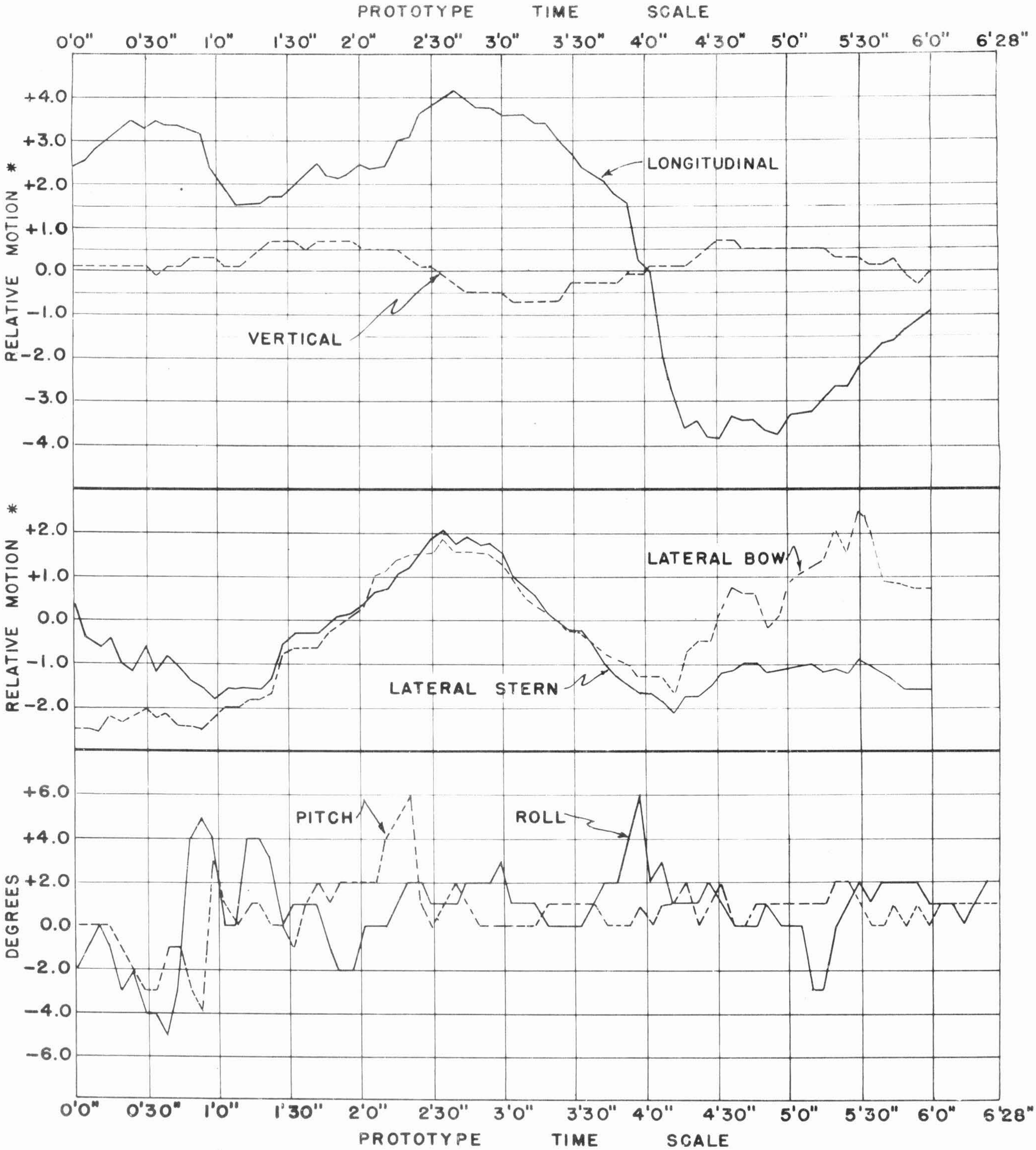


* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 3 MINUTE SURGE

FIG. 181 SHIP MOTIONS CAUSED BY 3 MINUTE SURGE WITHOUT MOLE IN PLACE - RESTRICTED TIES loose ties replaced by the restricted ties. The most striking effect of the restricted ties is that for both wave trains the amplitude of roll is greatly increased. This is a consequence of the fact that the tie lines are quite a distance vertically above the center of pressure of the water that acts on the hull of the ship. Thus, as the ship moves away laterally from the pier, if it reaches the limits of its lines before the horizontal motion of the water has reversed, the ship heels over due to the effect of rolling moment applied through the water and the lines. The restricted ties have no apparent effect on the amplitude of the pitch. For the fifteen second waves, however, the restricted ties seem to increase slightly the horizontal components of motion. This increase is of doubtful significance, since the amplitude of the motion as indicated by the loose ties is about of the same magnitude as the limited motion permitted by the restricted ties. In the case of the three minute surge, the horizontal components of motion are less with the restricted ties than with the loose ties. This is because the motion with the loose ties is greater than that permitted by the length of the restricted ties. However, the results of the study of the three minute surge with no ties as shown in Figure 182 indicates that the loose ties were exerting no restraint upon the ship, since the amplitude and character of the motions with the loose ties and with no ties are the same.

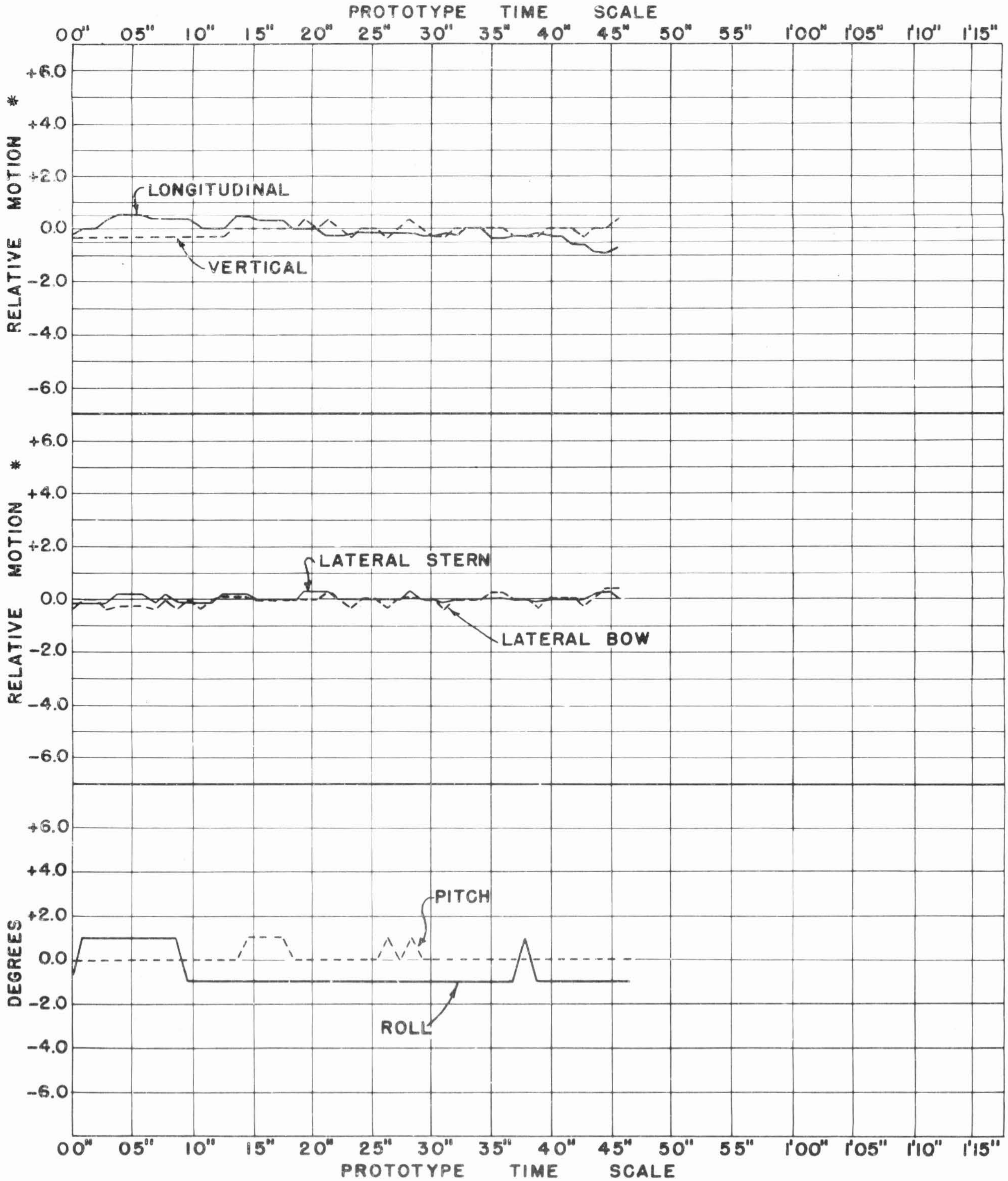
(4) Effect of mole and various gate openings on ship movements. The second item to be investigated in this series of studies was the effect of the presence of the mole on the amount of motion produced by the various wave trains. In making these studies the amplitudes and frequencies of the three standard wave trains were kept the same as they were for the basin without the mole. The units of motion used on the charts were also kept the same, i.e., the amplitude of the exciting wave train in the unmodified basin. Thus, the diagrams that present the results are all comparable. It is, of course, impossible to divorce the effect of the mole from that of the width of the gate. Therefore, these results will be presented together.

Figures 183 and 184 show the measured ship motions due to the fifteen second wave train and the three minute surge train, respectively. These results are for the mole with the 2070 ft. gate opening and with the ship fastened to the pier with loose ties. If these figures are compared with Figures 177 and 178, it will be seen that the effect of the mole with this wide gate is quite different for the two wave trains. For the fifteen second train it will be observed that the vertical ship motion with the mole and wide gate is reduced to 70% of its amplitude without the mole. The maximum amplitude of the longitudinal motion with the mole is nearly the same as it was without, but the motion appears smoother, i.e., the accelerations presumably are lower. The lateral motions appear to be reduced by about 70% and the pitch and roll by about 50%. If the comparable diagrams for the three minute surge trains are now examined, it will be seen that conditions are quite different. The vertical motion shows a reduction of about 30% with the mole in place. However, the longitudinal motion is still about six and one-half times the amplitude of the



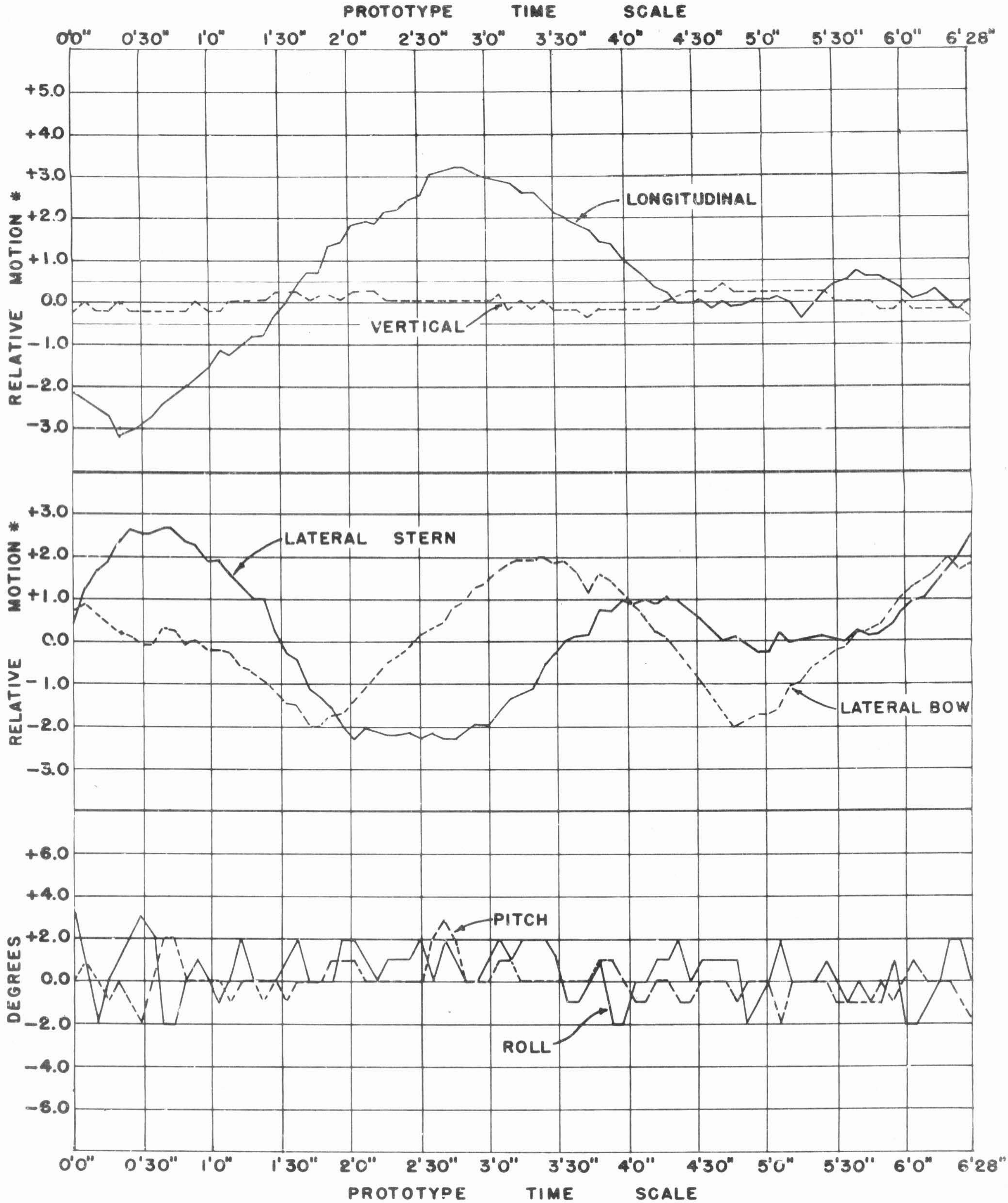
* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 3 MINUTE SURGE

FIG. 182 SHIP MOTIONS CAUSED BY 3 MINUTE SURGE WITHOUT MOLE IN PLACE - NO TIES



* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 15 SECOND WAVES

FIG. 183 SHIP MOTIONS CAUSED BY 15 SECOND WAVES WITH MOLE IN PLACE, 2070 FT. GATE OPENING, LOOSE TIES



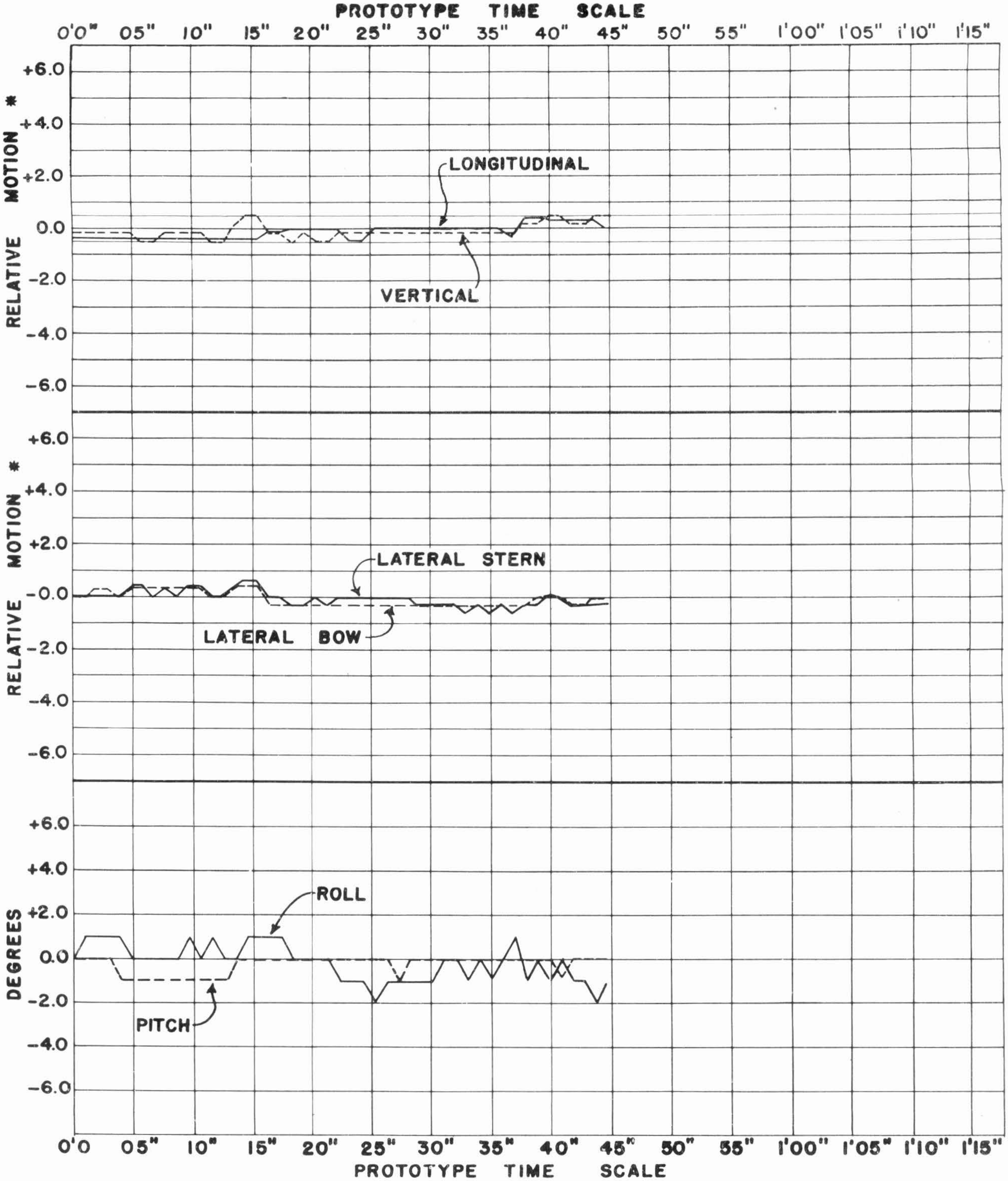
* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 3 MINUTE SURGE

FIG. 184 SHIP MOTIONS CAUSED BY 3 MINUTE SURGE WITH MOLE IN PLACE, 2070 FT. GATE OPENING, LOOSE TIES

surge outside the basin; whereas, without the mole, it was about seven and one-half times this amplitude. The lateral motions show no apparent reduction and neither does the pitch. The average roll is about the same with and without the mole, but the extreme peaks are reduced appreciably by the mole. On the other hand, there seems to be more high frequency component with the mole present. It will be seen from this comparison that, although the mole with the wide gate was reasonably effective in reducing the ship motions due to the fifteen second wave trains, it has practically no effect on those caused by the three minute surge trains.

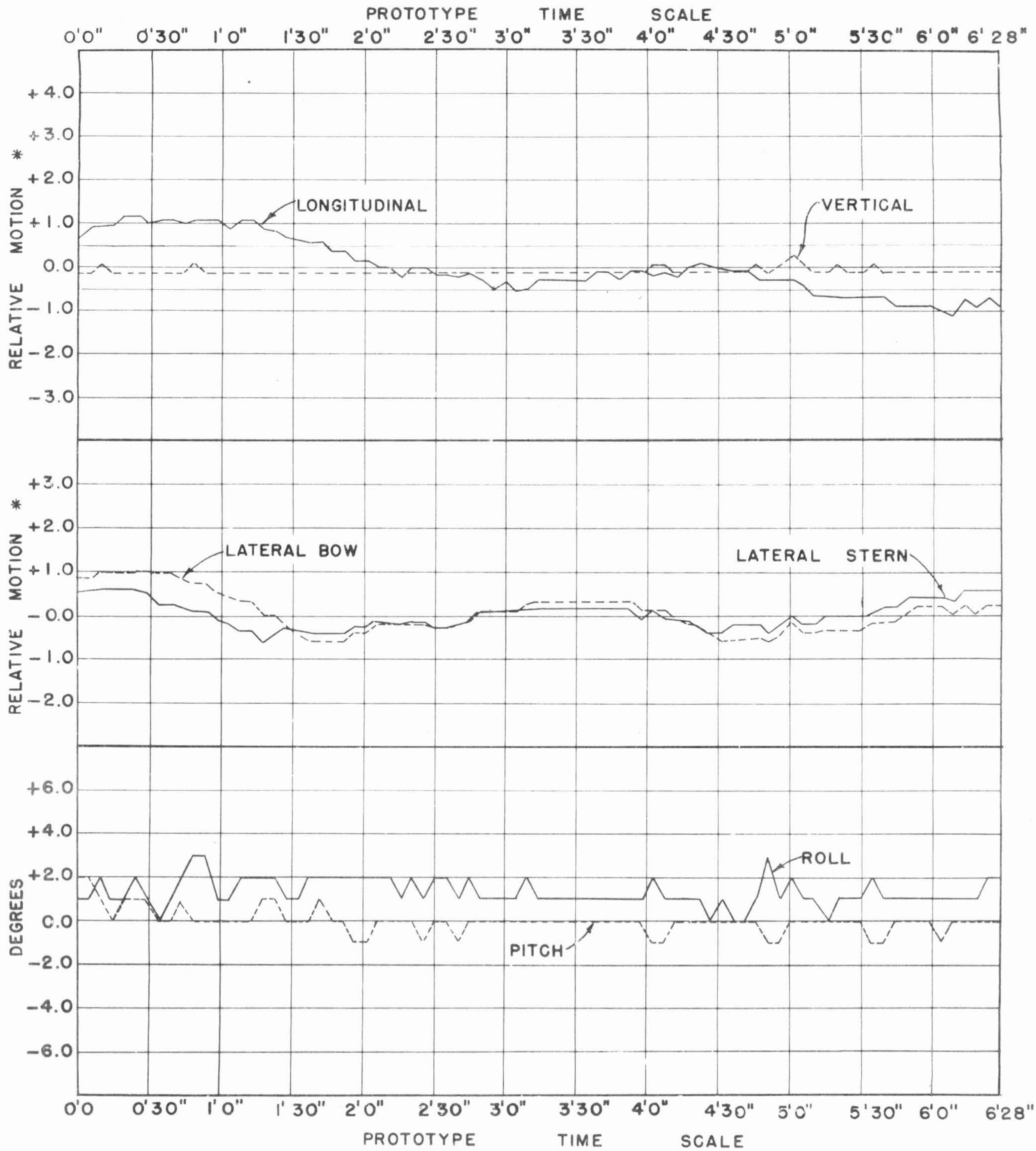
Figures 185, 186 and 187 show ship motions for a mole with a gate width of 750 ft., all other conditions remaining the same. If these diagrams are compared with the preceding sets, it will be observed that for the surge trains a substantial reduction in motion has been obtained by the use of the narrow gate. For the fifteen second wave trains, little reduction is noticeable between the two gate openings, although again the motion seems to be somewhat smoother. For the three minute surge train the vertical motion is down to about 50% of the amplitude of the undisturbed train. The longitudinal motion now has an amplitude of only about twice that of the original train instead of seven and one-half times with no mole and six and one-half times with the wide gate. The amplitudes of the lateral motions are down to about one and one-half as compared to six either without the mole or with the wide gate. The roll and pitch are also reduced significantly, being now about one-half what they were with the other two conditions. No diagram was available for the motion resulting from the six minute wave train with the mole and 2070 ft. gate. However, if the results for the 750 ft. gate are compared with those for the basin without the mole, it will be seen that some reduction has been obtained with the narrow gate. However, this is much less than was the case for the three minute surge. Thus, with the 750 ft. gate, the longitudinal motion is about 70% of that with the original basin. Approximately the same reduction is observed for the bow and stern motions. On the other hand, the roll seems to be about 50% of that for the modified basin. The characteristics of the pitch show little change. Figures 188 and 189 show the results of a further reduction of the gate from 750 ft. to 600 ft. width of opening. Once again, it will be observed that the motion due to the fifteen second waves shows little change in amplitude or in type. The motions with the three minute surge train also show little change when the gate is reduced to 600 ft.

In order to show more clearly the effects of these different gate openings as compared to the conditions with the unmodified basin, Figure 190 has been prepared, which shows all four lateral motions in one diagram for the conditions without the mole and with the mole with gate openings of 2070 ft., 750 ft., and 600 ft. As has been repeatedly stated in the report, the three minute surge is considered to be the most significant disturbing factor. These ship motion studies confirm the conclusion that



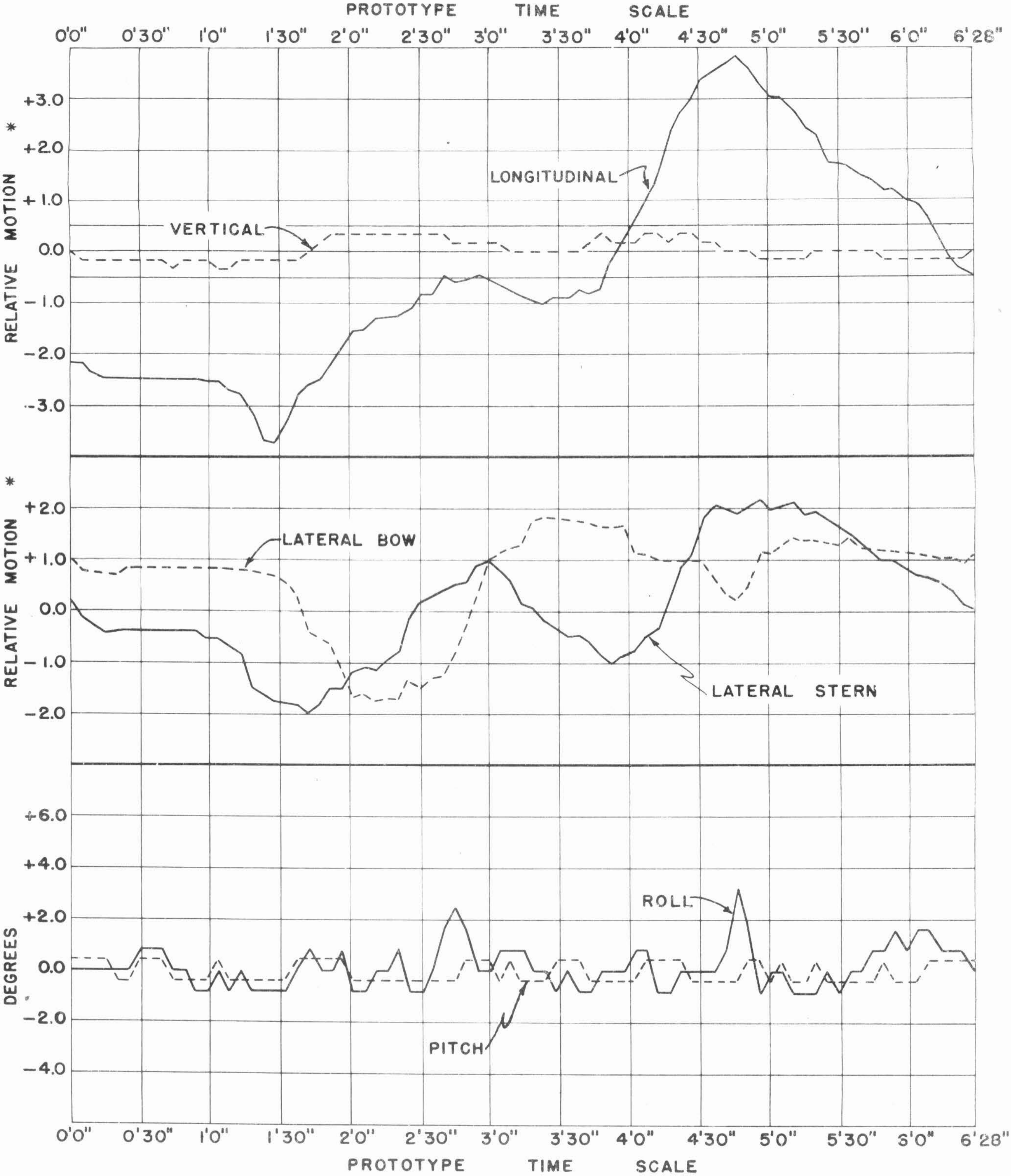
*NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 15 SECOND WAVES

FIG. 185 SHIP MOTIONS CAUSED BY 15 SECOND WAVES WITH MOLE IN PLACE, 750 FT. GATE OPENING, LOOSE TIES



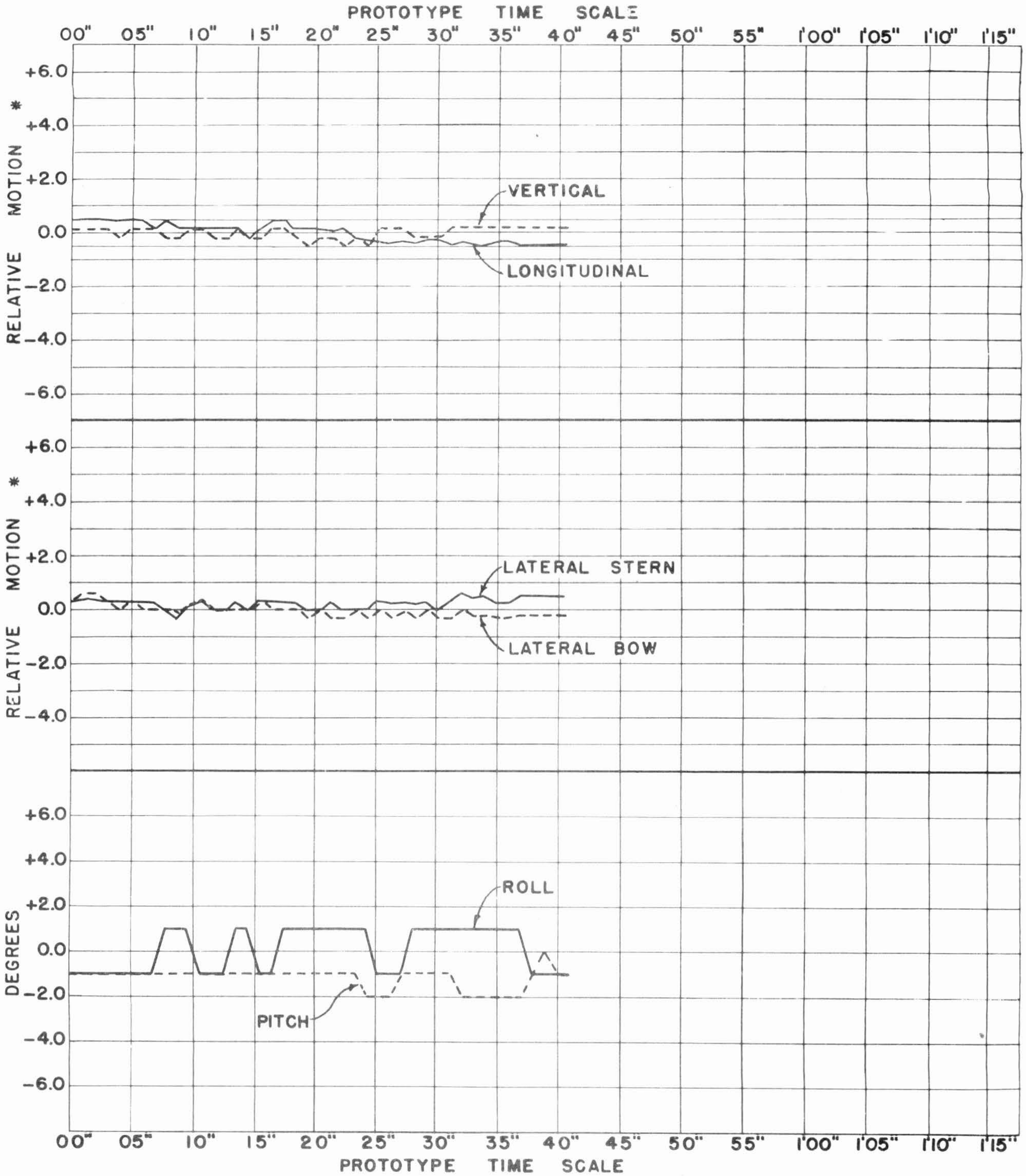
* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 3 MINUTE SURGE

FIG. 186 SHIP MOTIONS CAUSED BY 3 MINUTE SURGE WITH MOLE IN PLACE, 750 FT. GATE OPENING, LOOSE TIES



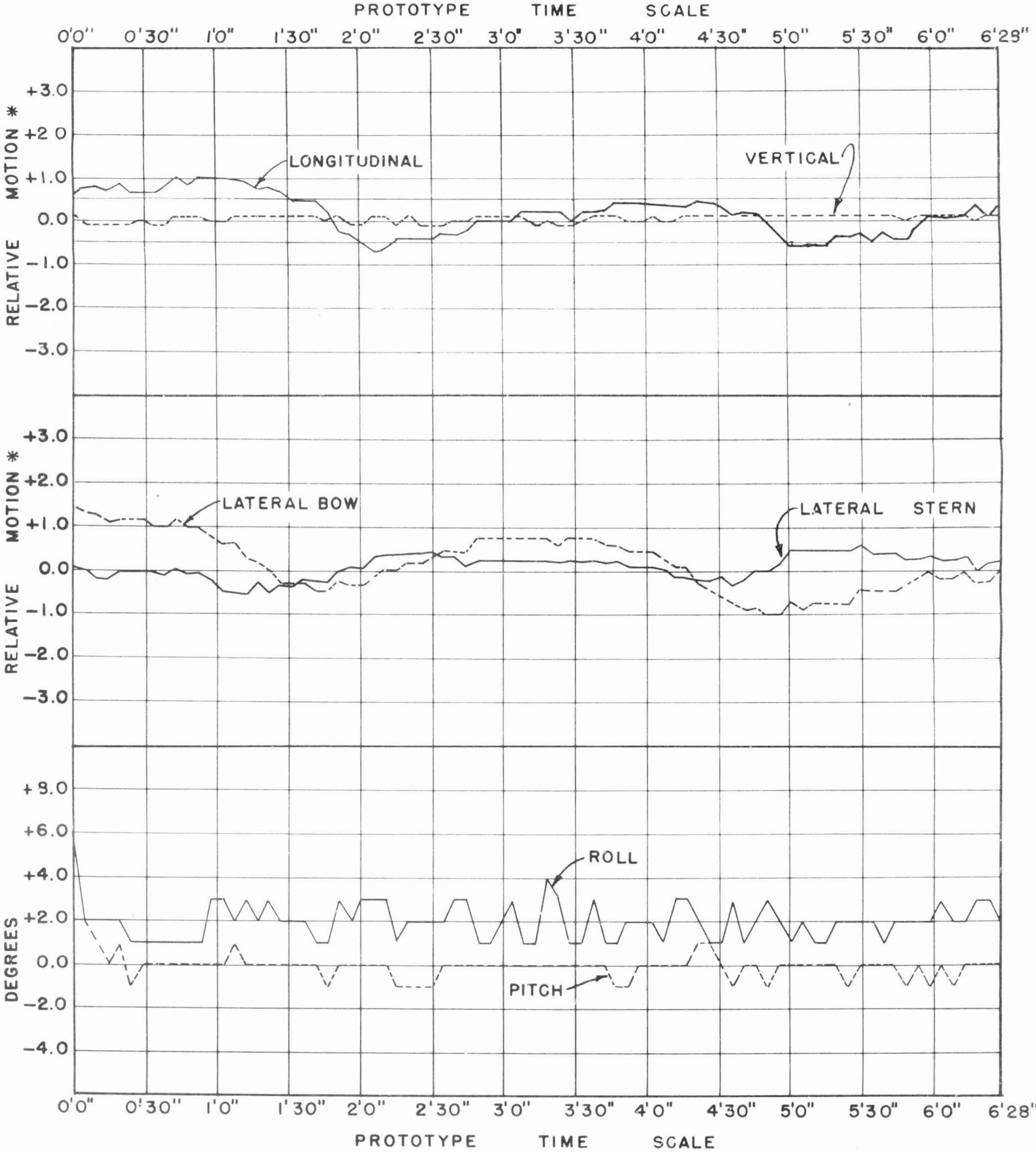
* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 6 MINUTE SURGE

FIG. 187 SHIP MOTIONS CAUSED BY 6 MINUTE SURGE WITH MOLE IN PLACE, 750 FT. GATE OPENING, LOOSE TIES



* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 15 SECOND WAVES

FIG. 188 SHIP MOTIONS CAUSED BY 15 SECOND WAVES WITH MOLE IN PLACE, 600 FT. GATE OPENING, LOOSE TIES



* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 3 MINUTE SURGE

FIG. 189 SHIP MOTIONS CAUSED BY 3 MINUTE SURGE WITH MOLE IN PLACE, 600 FT. GATE OPENING, LOOSE TIES

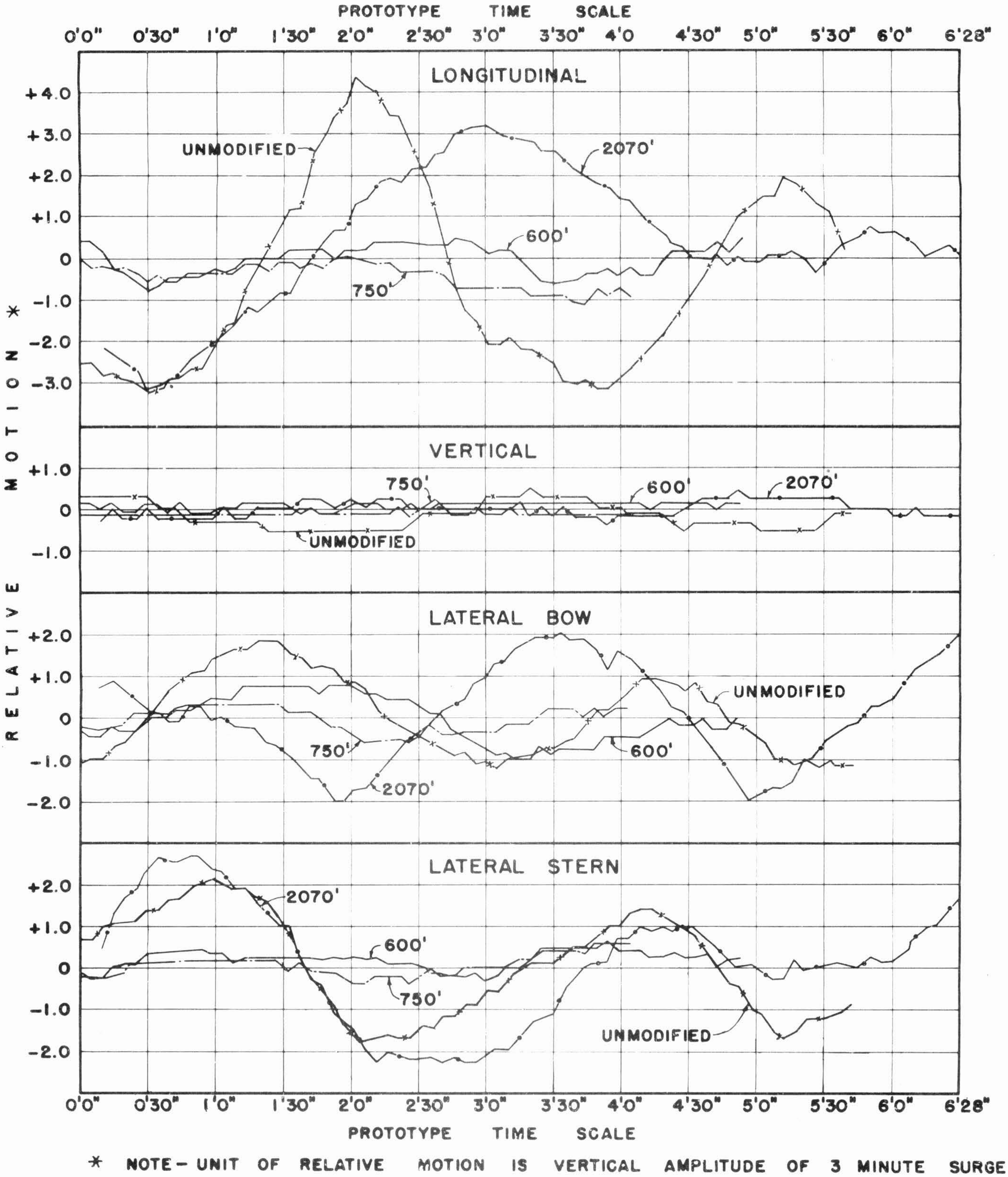
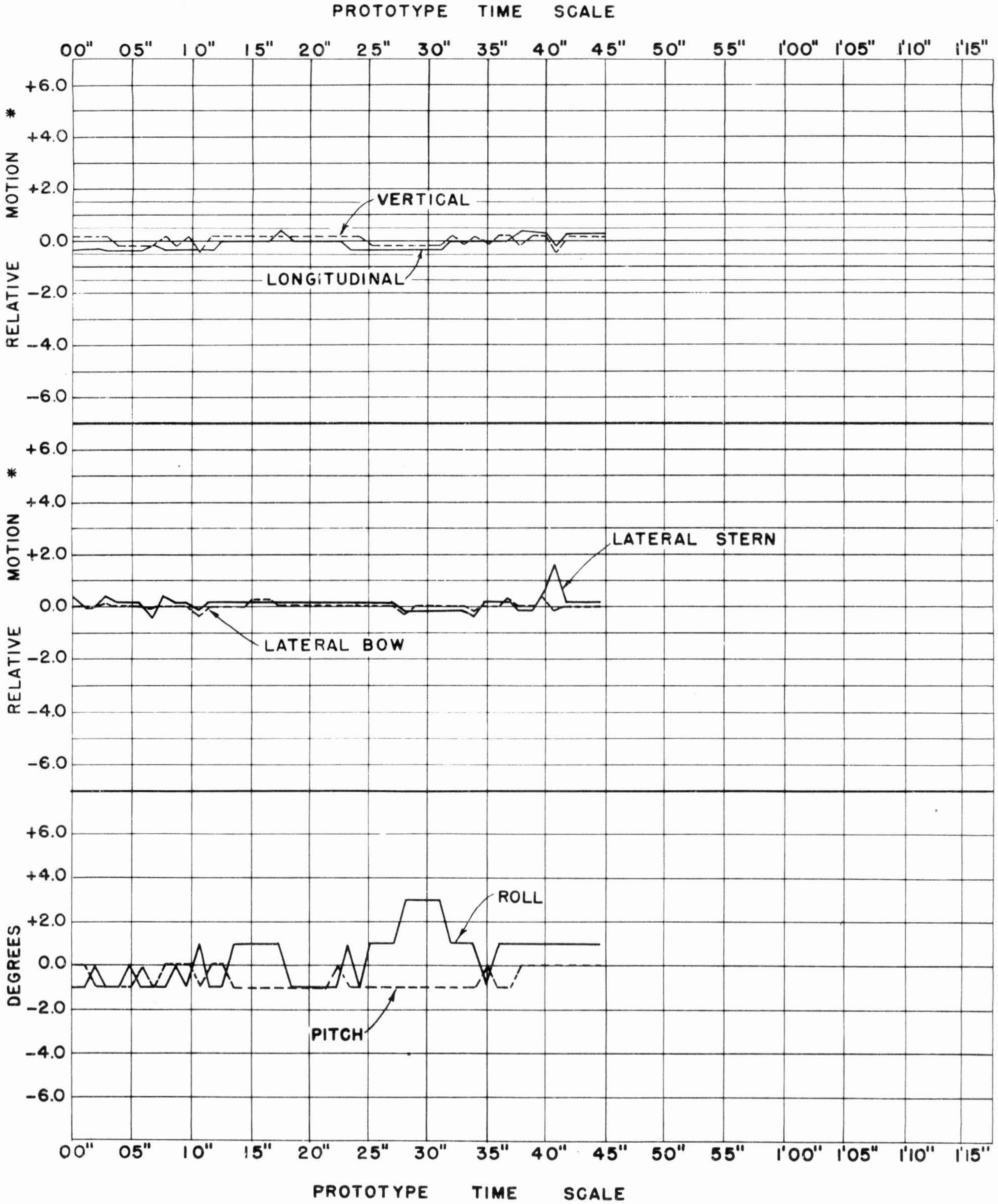


FIG. 190 SHIP MOTIONS CAUSED BY 3 MINUTE SURGE, UNMODIFIED BASIN, 600 FT., 750 FT., AND 2070 FT. GATE OPENINGS

has been reached from the investigation of the horizontal and vertical water motion; i.e., that in order to achieve any acceptable protection, a mole with not over a 750 ft. gate opening should be constructed.

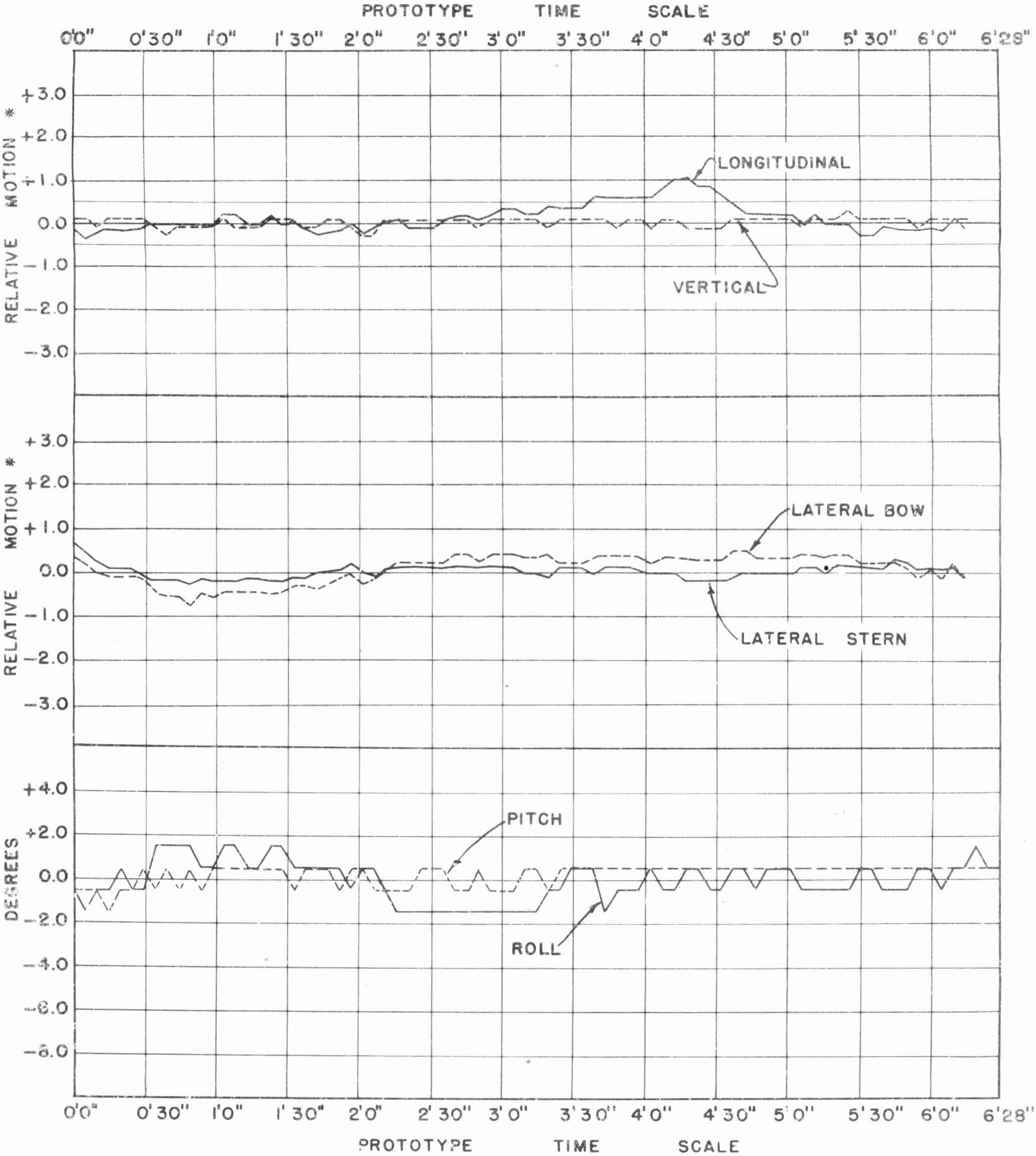
(5) Effect of opaque piers. One conclusion that was suggested by extended periods of visual observation of ship and water motions and one that is confirmed by analytical considerations is that the oscillating motion is one of the most undesirable types of ship movement. Steady currents, such as the drift currents induced by wave or surge trains or by steady forces which might be produced by wind, produce comparatively little effect since the ship moves to the limit of its travel in the direction of the force and remains there quietly. On the other hand, the oscillating movement of the water causes a similar oscillation of the ship. In particular, oscillations that are perpendicular to the piers are extremely undesirable and result in much damage. The basic thought behind all of the tests made with opaque piers is the elimination of the lateral oscillation of the water and hence of the ships. To investigate the effectiveness of this type of construction, ship motion studies were made for the 750 ft. gate opening with Piers 1, 2, 3 and 4 opaque. Figures 191, 192 and 193 show the results of these studies for the three different wave trains. In each case, the ship is fastened with loose ties. It will be observed that with both the fifteen second and three minute wave the lateral motion is reduced to a very small amplitude by the opaque piers. The motion from the six minute surge, however, shows only a rather small reduction. For the three minute excitations, the longitudinal amplitude is also significantly reduced by the opaque piers. It would seem that this reduction in motion is significant enough to warrant a careful consideration of the possibility of reconstructing the piers in the drydock area to eliminate the lateral motion of the water.

(6) Effect of geometry of wave pattern on components of ship motion. An examination of the various ship motion diagrams for the basin either with or without the mole shows that, although the periods of the different components of motion are all the same as the period of the exciting wave train, there is a difference in phase between the various components. Thus, for example, the extremes of motion of the longitudinal component occur at different times than do those of the lateral bow movement. Furthermore, the lateral stern movement in general does not synchronize with the lateral bow movement, which means that the ship is swinging in addition to its other motions. The reason for these differences in the phase is found in the geometry of the wave pattern. If the oscillatory motion of the water were always parallel with the piers, then it would be expected that the ships would simply have a longitudinal motion. If the water motion were perpendicular to the piers, then lateral motion of the ships would be expected, with the bow and stern moving together. However, if the oscillation is inclined to the pier, then both longitudinal and lateral motions can be anticipated. Furthermore, careful observation of the travel of the reflectors in the photographic studies of the horizontal water motion shows that, in general, the



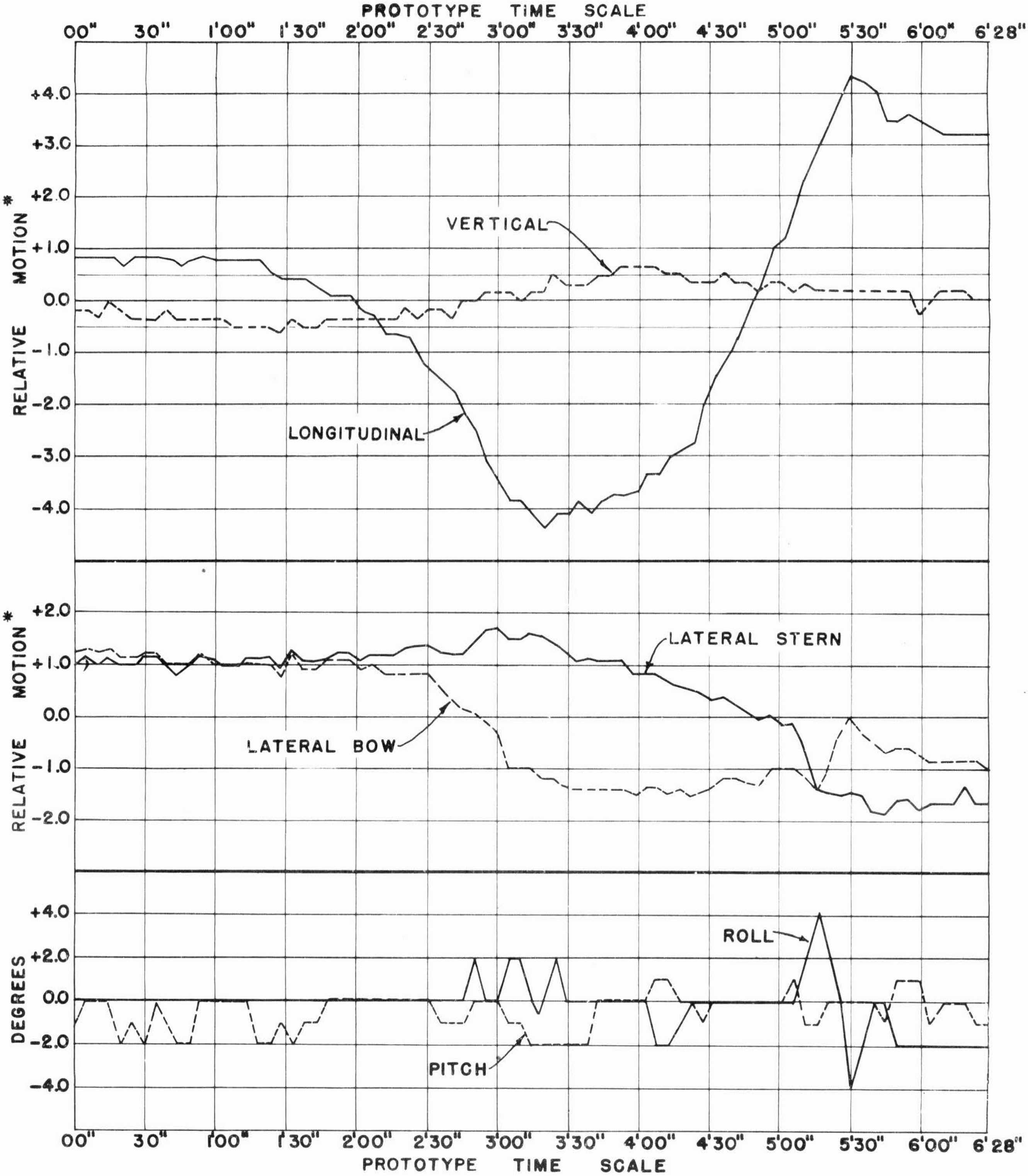
* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 15 SECOND WAVES

FIG. 191 SHIP MOTIONS CAUSED BY 15 SECOND WAVES WITH MOLE IN PLACE, WITH PIERS OPAQUE, 750 FT. GATE OPENING, LOOSE TIES



* NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 3 MINUTE SURGE

FIG. 192 SHIP MOTIONS CAUSED BY 3 MINUTE SURGE, WITH MOLE IN PLACE, WITH OPAQUE PIERS, 750 FT. GATE OPENING, LOOSE TIES



*NOTE: UNIT OF RELATIVE MOTION IS VERTICAL AMPLITUDE OF 6 MINUTE SURGE

FIG. 193 SHIP MOTIONS CAUSED BY 6 MINUTE SURGE, WITH MOLE IN PLACE, WITH PIERS OPAQUE, 750 FT. GATE OPENING, LOOSE TIES water motion at any given point is not a simple oscillation, but is a rather complicated motion. For example, Figure 194 is a large scale reproduction of the movements of the floats in the basin because the water movements are greater and the complexity of the pattern is more apparent. It will be observed that the type of oscillation changes very significantly from place to place in the basin and, furthermore, even for one float the type of oscillation may vary from the beginning to end of its observed travel. Thus, for a good-sized ship, the type and phase of the oscillation at the bow may be quite different from what it is at the stern.

(7) General comments on ship motions. This section of the model study has demonstrated quite clearly how closely the ship motions follow the water motions. Since, for a given vertical amplitude of wave motion, the amplitude of the accompanying horizontal motion increases very rapidly with the period and wave length, it follows both analytically and experimentally that long period wave trains whose vertical amplitudes are so small as to easily escape

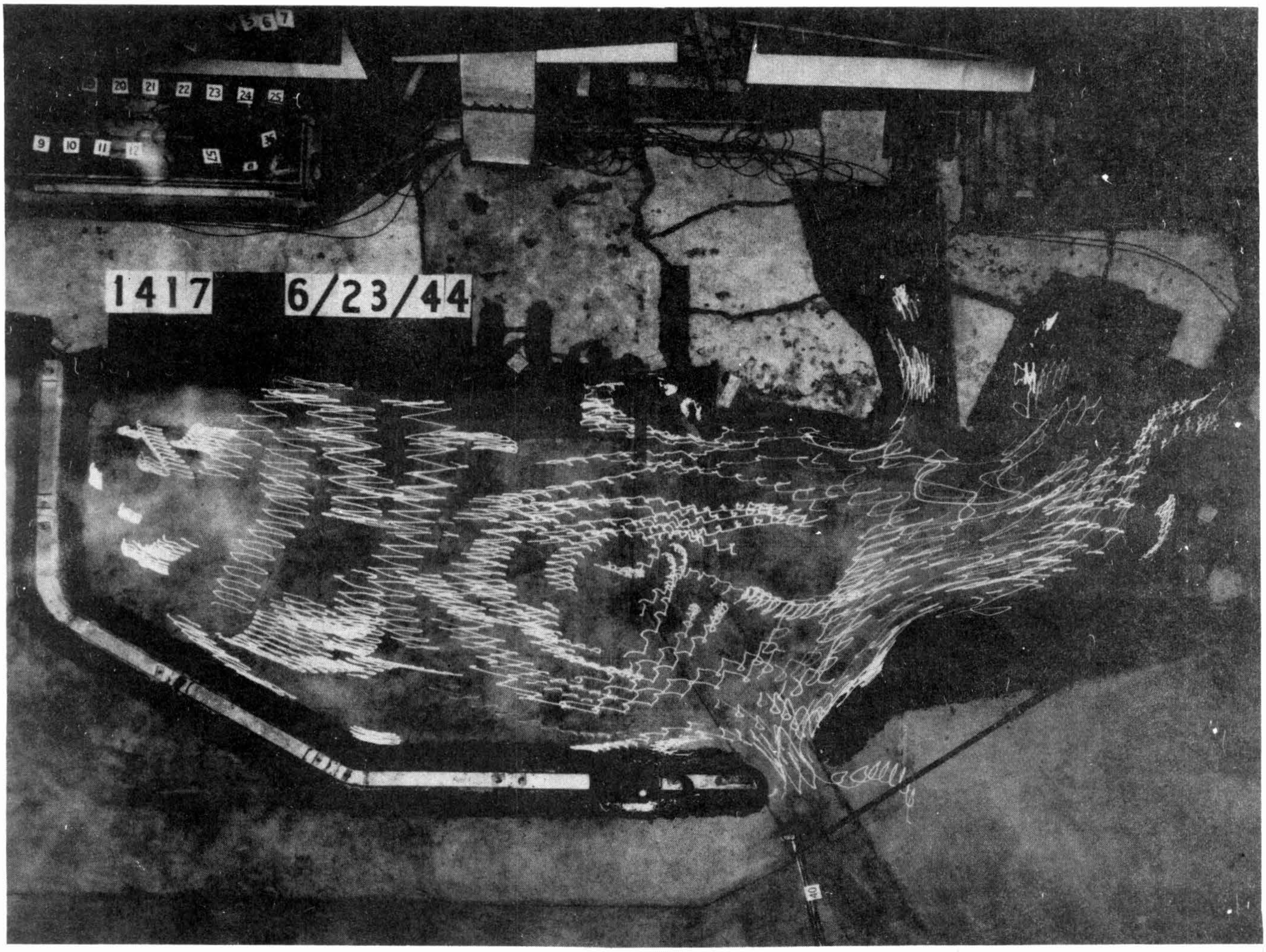


FIG. 194 FLOAT MOVEMENTS RESULTING FROM SIX MINUTE SURGE

observation produce horizontal ship motions of damaging magnitude. It has been seen that the steady current, or drift current, which is produced as a secondary effect by the waves and surge, has comparatively little undesirable effect but that the oscillating component of the water motion is the element that must be eliminated if a satisfactory basin is to be obtained.

(h) Channel damping. It will be remembered that in Section V-C it was pointed out that with shallow water waves a dredged channel seemed to be effective in damping the magnitude of a wave train whose direction of travel paralleled that of the channel. Figure 38, Page 60, which was made in the ripple tank, shows evidence of this effect. This same damping was observed visually on both Model 2 and Model 3 in the model basin. In order to investigate this effect more quantitatively, special runs were made in the basin without the mole, in which the measurements of the vertical motion were extended seaward from the basin area towards the wave machine as far as conditions would permit. These measurements were made with two conditions of wave motion; first, with the fifteen second wave train originating at the east gate only and second, with the two fifteen second wave trains, one from each gate. No comparable measurements were made for the three minute surge because, as has been pointed out, in order for this type of damping to be effective, the length of the travel of the wave along the channel has to be equal to several wave lengths. The dredged channel from the east gate to the Naval Operating Base was sufficiently long to fulfill this criterion for the fifteen second waves since the length of the channel is about 8,000 ft., which is roughly 15 times their 500 ft. to 600 ft. wave length. On the other hand, the three minute wave length is about 6500 ft. in this depth of water, which means the channel is only about one wave length long; hence, little damping could be expected due to the action of this channel on the three minute and longer surge waves.

Figures 195 and 196 show direct photographs of the wave trains for the conditions just outlined. It will be seen that in Figure 196, which shows the waves originating from the east gate only the height of the section of the wave crest that travels along the deep channel decreases more rapidly as the Naval Operating Base is approached than do the heights of the crests on either side of the channel. In Figure 196, which shows the waves coming from both gates, this reduction in the waves from the east gate is still visible but the wave crests originating at the west gate travel across the channel with no visible change. Figures 197 and 198 are the two contour maps of the vertical amplitudes of motion for these same tests. These two maps show clearly the progressive nature of this damping. Thus, in Figure 197 it will be observed that in the portion of the channel nearest the east gate, the amplitude of the vertical motion is about the same in the channel as it is on either side of it; whereas, as the waves approach nearer and nearer the drydock area, the difference in amplitudes of the wave crests within the channel and on either side of it becomes larger and larger. When the waves have reached the location

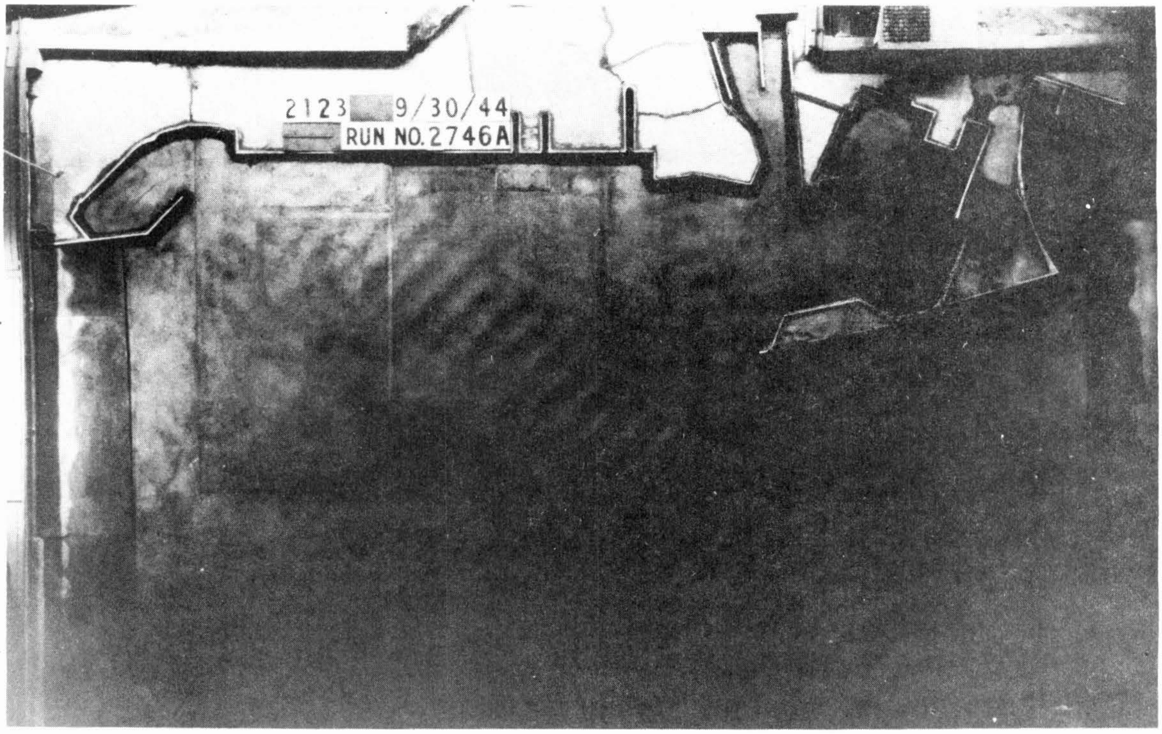


FIG. 195 EFFECT OF CHANNEL ON 15 SECOND WAVE TRAIN FROM EAST GATE

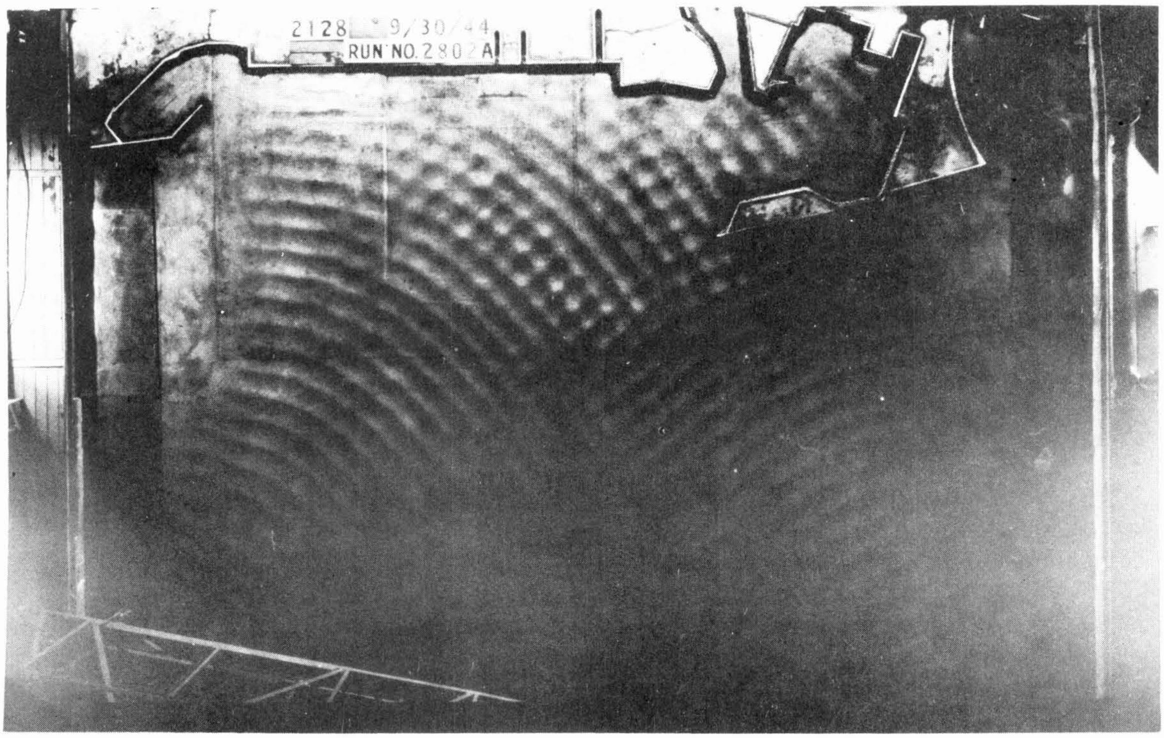


FIG. 196 EFFECT OF CHANNEL ON 15 SECOND WAVE TRAIN FROM BOTH GATES

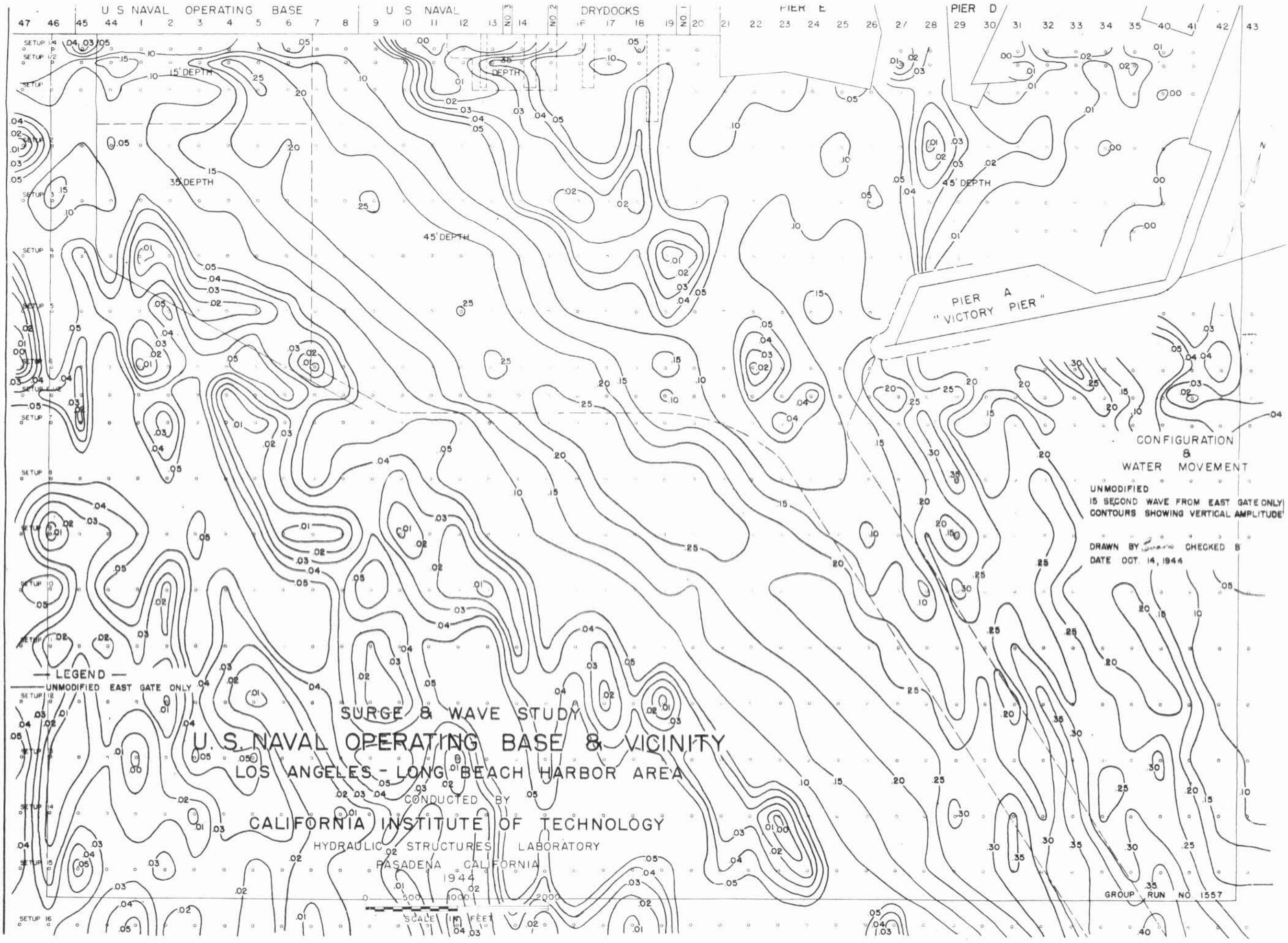


FIG. 197 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES FROM EAST GATE ONLY

of the mole entrance the reduction is about 50%. In Figure 198, with waves coming from both gates, the same tendency is observed, although the standing wave pattern is quite different. However, the similarity holds good only until the wave reaches the mole gate. With both wave trains entering the area there is apparently a region of rather high vertical motion just west of Pier A. This is due probably to the reflection from the end of Pier A of the wave train that originates at the west gate. It will be observed that the motion in this same area due to wave trains from the east gate is very small.

It must be reemphasized that this phenomenon is one that is peculiar to shallow water waves because for its operation it depends completely on the variation of the wave velocity with the depth. Therefore, it will have no effect on surface chop or any other waves which have a wave length of less than, say, 200 ft. For example, if this model study had been made with a larger vertical distortion factor, the results would have shown no indication of this damping.

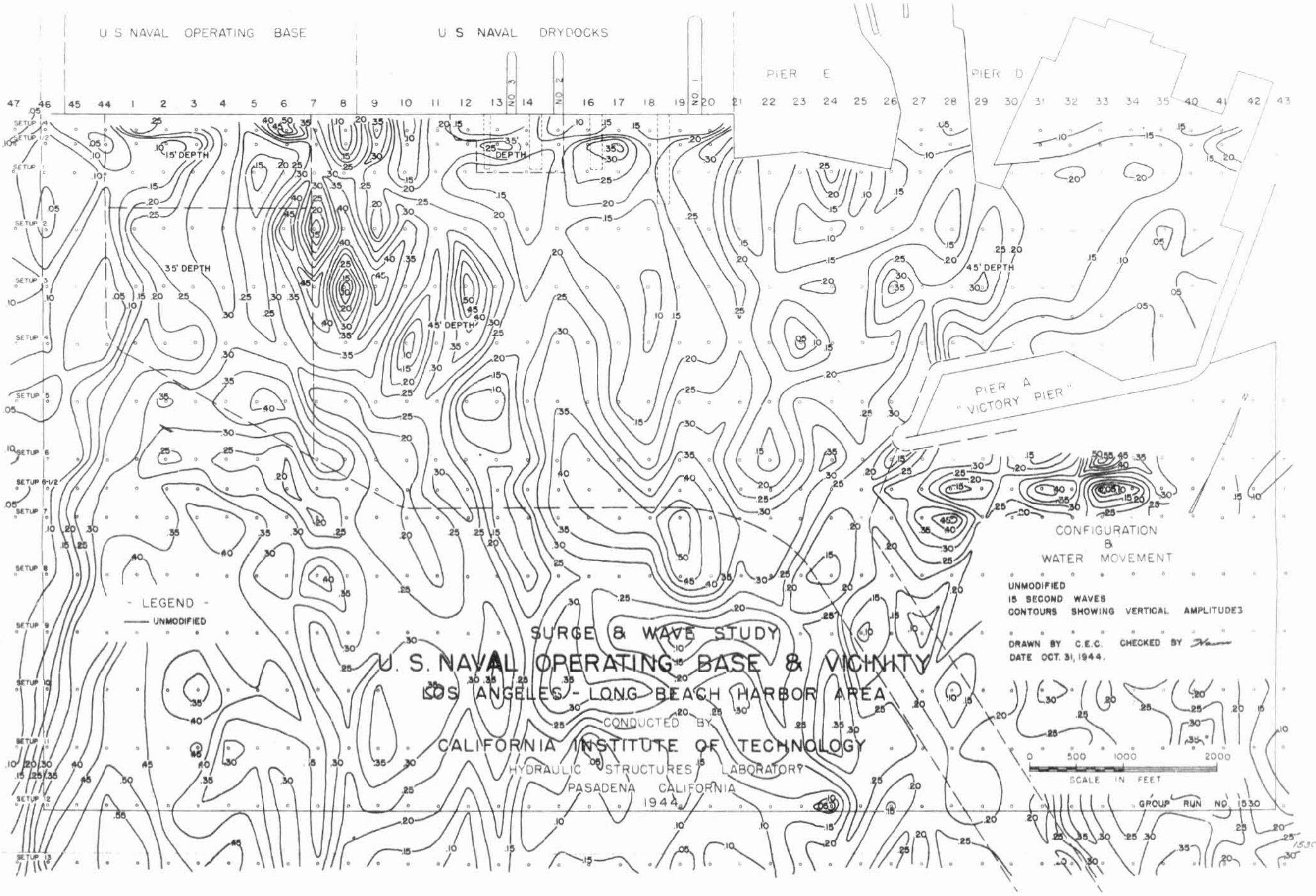


FIG. 198 VERTICAL MOVEMENT CAUSED BY 15 SECOND WAVES FROM BOTH GATES

VIII. DISCUSSION, CONCLUSIONS, AND RECOMMENDATIONS

A. DISCUSSION OF VALIDITY OF RESULTS

During this model study, three general types of experimental determinations have been made both in the basin without the mole and the basin enclosed by the mole.

(1) The delineation of the traveling and the standing wave patterns.

(2) Measurement of the amplitude of the vertical and horizontal water motions.

(3) Measurement of the components of motion of model ships.

The following brief discussion of the validity to be expected of the results of this study analyzes separately these three types of measurements.

1. STANDING WAVE PATTERNS

In the basin without the mole, the principal short period standing wave pattern is produced by the interference of the waves from both gates together with the reflections of these trains from the solid boundaries. For the fifteen second wave trains, the relative phases of the trains from the two gates are not known. Therefore, it is possible that the wave pattern could be displaced in either direction by about one-half a wave length, i.e., by about ± 300 ft. This means that small discrepancies between the pattern found in the model and the pattern found in the harbor can be expected, but probably not large ones. However, it should be remembered that the standing wave pattern is greatly affected by the changes in the period of the trains. Therefore, if the wave period at the harbor deviates appreciably from the twelve to fifteen second period used for the model study, corresponding changes in the wave patterns are to be expected. In the case of the three minute surge, or surges of longer period, model and prototype results should agree even more closely because there is probably only one wave train with its resulting reflections in operation to form the standing wave pattern.

With the mole in place, conditions within the basin are simplified because with the single gate, there is only one source of disturbance possible. Thus, the wave pattern at the harbor should agree very closely with that found in the model, since in general the pattern is determined only by the basin configuration, the gate location, and the wave frequency.

2. AMPLITUDES OF THE VERTICAL AND HORIZONTAL WATER MOTIONS

The discussion of the validity of the model measurements for the vertical and horizontal water motions reduces itself to a comparison of the amount of damping in the model to that in the prototype. It is undoubtedly true that in the small-scale models used for this study the damping is considerably greater than it is in the prototype. Therefore, conclusions based on scaling up model results obtained in a single test give optimistic results as to the protection afforded by a given type of mole construction. For example, if a single test were made with the recommended mole in place with the 750 ft. gate opening, it would be dangerous to compare the vertical and horizontal motions observed in the outer harbor with those at Pier 1 for the purpose of determining the protection afforded by the mole. Instead of obtaining the information in this manner, these results in the model study have been secured by making a comparison of the motions measured at Pier 1 with no mole in place with those observed at the same point with the mole installed with a 750 ft. gate opening. These comparisons are made for standardized wave and surge trains. This method of approach eliminates to a large extent the excess damping of the model. Therefore, the results as presented should be a reliable indication of the behaviour of the prototype. There may be one exception to this statement. The horizontal motion shows two components - an oscillation of the same period as that of the exciting wave train and a current or drift. These drift currents are so small that they are susceptible to minute outside influences in the model and, therefore, the current pattern observed in the model may not agree with that existing in the prototype. However, these drift currents seem to be of little significance in the present problem. The horizontal oscillations, however, should have the same validity as the vertical oscillations and are, therefore, to be considered reliable for the study.

3. SHIP MOVEMENTS

The question of the validity of the movements of the model ships is largely one of the effect of the vertical distortion of the harbor model since the ship models are made without distortion. Aside from this discrepancy, there should be no question, since the model ships are geometrically to scale and have the correct rolling and pitching periods. The fact that they float with the correct waterline means that their weight is to scale with their shape. Therefore, if it were not for the discrepancy in this distortion factor, the ship models should have the same degree of similarity with the prototype as does the harbor model and, therefore, results obtained from studying the ship models should be as valid as any other results of the test. It would seem at first that the lack of vertical distortion of the ship models would result in the presentation of less area to the horizontal water motion and, hence, the model ships should not follow the water movements as closely as do the ones in the harbor. However, if the model ships had been made with the vertical distortion they would not only have presented twice as much area to the horizontal water motion but they would have had twice the mass and, therefore, require twice the force to produce the same movement. Furthermore, since all of the waves with which this study is concerned are shallow water waves, there should be little variation between surface and bottom velocities at any given point. It would seem, therefore, that the discrepancies, if any, must be due to secondary effects, such as viscosity, surface tension, etc. A comparison of the available evidence from the model study and from the observations at the harbor shows that in both cases the ship motion agrees closely with the water motion. Therefore, it is believed that the ship motion studies are valid indications of the prototype performance.

B. CONCLUSIONS

The first seven conclusions summarize the field conditions that are believed to be causing the undesirable water movements and the damage to ships, piers, and drydocks. They represent the results of field measurements, analyzed and interpreted with the assistance of the observations in the model.

(1) The normal condition seaward of the outer breakwater is to have waves approaching from the southwest with their crests approximately perpendicular to the reach of breakwater at the San Pedro lighthouse. During storm conditions, waves may approach the breakwater from the south or southeast.

(2) Observations show that, although fifteen second waves cause some flow through the outer breakwater, the motion is damped very appreciably indicating that this breakwater offers good protection from the waves of this period. However, the fact that any flow comes through the breakwater under the action of the fifteen second waves indicates that damaging waves with periods of approximately three minutes can come through the outer breakwater, although the attenuation probably is appreciable.

(3) Due to the porosity of the outer breakwater, and also to the presence of the navigation openings and the open east end, the outer breakwater does not provide complete protection to the outer harbor. Although very little disturbance, due to the 15 second wave trains, passes through the breakwater, field observations show that these waves, having lengths from crest to crest of about 600 ft., enter both navigation gates and spread in concentric rings in the outer harbor area.

(4) Field observations show that wave trains with periods varying from about 10 seconds to one hour occur in the drydock area. Fifteen second and three minute periods are common. Both field and model results show that the three minute surge waves are responsible for major ship movements and damage. More specifically, surges with periods of from one to three minutes which have amplitudes of two-tenths of a foot or greater will induce ship

motion capable of causing appreciable damage. On the other hand, the fact that the drydock gates were handled during such surge conditions without difficulty indicates that surge motion does not interfere seriously with this phase of drydock operation.

(5) Very little, if any, damage occurs to ships and piers during times of abnormally high waves which have periods of about fifteen seconds. Conversely, the evidence indicates that these fifteen second wave trains do interfere with the operation of the drydocks and cause damage to the gates and seats.

(6) Damaging surges are usually, but not necessarily, accompanied by high 15 second wave trains. On the other hand, high waves of 15 second periods often occur without observable surge waves of one minute or longer periods.

(7) The outer harbor basin is too small and too intimately connected to the ocean to have appreciable independent water movements. Therefore, disturbances must originate outside of the harbor and enter the harbor either through the navigation gates, the open end, or through the breakwater itself.

The remaining conclusions present the results of the model study.

(8) A properly designed mole will give adequate protection to the entire inner basin from the effects of 15 second wave trains and less complete protection from one minute and longer surge trains.

(9) Neither of the designs proposed by the Navy, namely, the "Original Mole" or the "Standard Mole" is capable of affording the required degree of protection to the inner basin without significant modifications. The major change required is the reduction of the gate opening from 2070 ft. to 750 ft. However, appreciable improvements also result from modifying the mole alignment and altering the exact location and shape of the gate opening.

(10) The reduction of the gate opening in the mole is particularly necessary to reduce the amount of surge-produced movement in the inner basin. The entire mole and Pier A extensions at the gate must be of tight, or opaque, construction to be impervious to the effects of one minute and longer period surges.

(11) The mole with the 2070 ft. gate opening affords considerably more protection from the short period waves than from the one minute and longer surges. However, even for the short period waves, the reduction of the opening to 750 ft. results in a significant improvement of conditions within the basin.

(12) The portion of the basin enclosure which is known as Pier

A extension, and which forms an integral part of the mole system, should be constructed in accordance with the findings of this study. It forms the eastern side of the navigation gate opening and its shape, alignment and degree of permeability all affect the performance of the basin.

(13) The replacement of the right-angled corner used in the "Standard Mole" design by a reach of mole inclined at 30° to the shoreline (somewhat similar to the construction proposed in the "Original Mole" design) results in the reduction in the amplitude of the short period wave motion along the drydock shoreline.

(14) Additional structures within the mole area which increase damping and add wave paths improve the general condition in the basin.

(15) The construction of Pier A wharf has apparently slightly increased the motion in the basin. The explanation is that probably it has reduced the damping effect by smoothing the shoreline, thus reducing the number of reflected waves set up in the area.

(16) Deepening of the major part of the basin appears to improve conditions slightly within the area enclosed by the mole.

(17) Uniform conditions do not prevail over the entire basin. Vertical amplitude maps indicate locations of "live" and quiet areas. Typical live areas are found in the northwest corner of the basin, along the inner side of the mole near the gate, and also along the south side of Pier E extension.

(18) The shape of the seaward side of the mole ends is important to avoid the possibility of "funnel action" which can increase the effective gate width and, hence, increase the disturbance in the inner basin.

(19) The mole width is important only as it affects the configuration of the basin or of the gate ends.

(20) The motion of the ships follows closely the horizontal and vertical motion of the water for three minute and longer period surges. It does not follow the horizontal motion of the fifteen second wave trains, although it follows the rise and fall.

(21) The lateral components of the ship motion at piers can be reduced greatly by making the piers opaque and thus eliminating the cross currents produced by the surges. However, the location of the piers to be opaqued, and the amount of opaquing that would produce the optimum results, is beyond the scope of the study.

(22) The effective fundamental basin resonance period is approximately six minutes. Wave trains of this period produce disturbances having the greatest magnifications. The motion within the basin with this train has an amplitude as large, and in some areas slightly larger, with the mole in place than without it. The occasional occurrence of surges of this six minute period must be anticipated.

(23) A deep channel which is normal to the wave crests of shallow water wave trains causes appreciable damping of the amplitude of the section of the wave crests lying in the channel. The channel from the east gate of the breakwater to the mole gate acts in this manner, thus reducing the effective disturbance from the waves which enter this breakwater gate. If the mole gate opening does not exceed the channel width, the disturbance within the mole area produced by this wave train is reduced considerably. This simple phenomenon appears to offer important possibilities for improving harbor design.

(24) It would seem possible to protect the entire outer harbor from the disturbances produced by the fifteen second wave trains by modifying the existing breakwater gates. However, such a development probably would not affect the disturbances due to surges having periods longer than one minute.

(25) Although many valuable field measurements have been made, the total amount of reliable information concerning the over-all conditions in the harbor area is still completely inadequate to serve as a basis for technically sound developments. The mole under consideration is a typical example. To a considerable extent, the conditions to be investigated in the model study had to be established on the basis of experience and judgment, rather than on factual data obtained from quantitative measurements. Therefore, in order to insure the reliability of the results, a much more extensive model study has been required than would have been the case if the field conditions had been known more precisely. Adequate field measurements are difficult to obtain. The existence of a series of standing wave patterns for the different wave trains introduces a serious complication. Due to this situation, records from a single station are of limited value. Such a record gives the summation of all the factors producing the disturbance at that particular location, but does not furnish sufficient information to permit the breaking down of the disturbance into its component parts, including their amplitudes, periods and directions of travel. To do this requires a set of carefully selected synchronized stations. All of the qualitative evidence indicates that surges with periods of the order of three minutes cause the major portion of ship and pier damage. However, quantitative measurements of their vertical am-

plitudes are very fragmentary and of their horizontal amplitudes and velocities are non-existent.

C. RECOMMENDATIONS

The first four recommendations deal directly with the construction of the mole.

(1) It is recommended that the alignment of the West or main mole be made in accordance with Figure 137, i.e., with two arms at right angles, one perpendicular and one parallel to the shoreline, connected with a diagonal arm 3,000 ft. long inclined at 30° to the parallel arm.

(2) It is recommended that the gate opening be restricted to 750 ft. width, toe to toe of slope. This opening should be centered on the existing 45 ft. deep channel from the east breakwater gate to the drydocks area. The plane of the opening should be perpendicular to the channel.

(3) It is recommended that the mole ends be shaped so that the minimum opening occurs on the seaward side. Any rounding desired should be confined to the shoreward side. Recommendations (2) and (3) are also shown on Figure 137.

(4) It is recommended that the entire mole system, including Pier A extension, be of tight or opaque construction that is impervious to long period surge motion.

The remaining recommendations concern future developments in the basin and the outer harbor areas.

(5) It is recommended that, in the planning of any additional construction within the basin, due consideration be given to the existing disturbance pattern:

(a) To determine the suitability of the site from the standpoint of the local disturbances and their effect on the proposed activity.

(b) To estimate the effect of the proposed construction on the disturbance pattern in the vicinity of other critical areas.

Such information is presented in this report, paragraph (e), Pages 103 to 108, for specific designs and locations of new drydocks, marginal wharf and piers, etc., which have been proposed by the Naval Operating Base.

(6) It is recommended that the use of opaque piers be studied, both for existing and for new construction, as a means of further reducing ship motion and the resulting damage both to the ships and the piers.

(7) It is recommended that an analysis be made of the action of the fifteen second waves in the outer harbor and, in the event that this action proves to be objectionable in other important areas, that consideration be given to the modification of the breakwater gates, to eliminate the movements from this source.

(8) It is recommended that a systematic study be made of the use of channels to effect wave damping and thus improve conditions in this and other harbors.

(9) It is recommended that a continuing program of field measurements and studies be initiated under competent technical research direction, independent of the operational activities of the Base, and that the scope of such studies should include:

(a) Determination of amplitudes, periods and direction of travel of the various existing wave trains.

(b) Determination of the standing wave pattern produced by these trains.

(c) Measurements of the horizontal amplitudes and velocities of the surge currents, and correlation of this information with the corresponding vertical amplitudes.

(d) Study and measurement of ship motions under normal, surge, and storm conditions, and correlation of these motions with the horizontal and vertical water movements.

APPENDIX I - DESCRIPTION OF RIPPLE TANK

The ripple tank is a laboratory instrument for the study of two-dimensional problems involving reflection, diffraction, spreading and dissipation of waves with essentially constant wave velocity in irregularly shaped areas. If the wave velocity in the harbor were constant, the ripple tank would allow the construction of an ideally small model. Because of the difficulty of reproducing the variable depth in the ripple tank, only a reasonable approximation of the wave pattern can be achieved and the results of a ripple tank study are essentially qualitative.

The detail of the frame work and the arrangement of the mirrors can be seen in Figure 26. The frame must be built as rigid as possible in order to prevent the creation of undesired waves in the tank due to vibrations of the frame. The tank which was used in this study was not completely satisfactory in this respect as whole systems of small waves appear in many photographs which distinctly follow all shorelines independent from the main pattern. These little waves are created by vibrations due to the movement of the wave machine used in the first model. In the second model, waves were made with a hand plunger which did not have sufficient mass to cause the frame to vibrate.

The main optical system consists of a point light-source, the glass-bottomed tank, and the screen. The mirrors only serve the purpose of folding the system so it can be housed in a normal room. (See Figure 27) As the result of experimentation, the following light sources were found to be most satisfactory. For direct observation a General Electric photomicrographic light bulb (30 amperes, 11 volts) was used, which concentrates the whole light output into a circle of less than one-eighth inch diameter. For photographic purposes the intensity of the light was not sufficient and it was replaced by an open air spark, created in a one-eighth inch gap, discharging the energy of a 10 microfarad condenser charged to 8500 volts.

From either point source the light spreads in all directions. A suitable screen eliminates all of the rays except those having a cone of proper size to illuminate the ripple tank. This cone enters the tank through the plate-glass bottom, proceeds through the water, leaves the tank through the water surface and is reflected by the front surface mirror, M_2, onto the screen. The mirror, M_1, can be a normal rear surface mirror since it is ahead of the ripple surface and, therefore, cannot affect the quality of the reflection on the screen.

A brief explanation of the optics of the system may help the interpretation of the photographs. Consider a single light ray from the point source, as shown in Figure 199. Assume that it

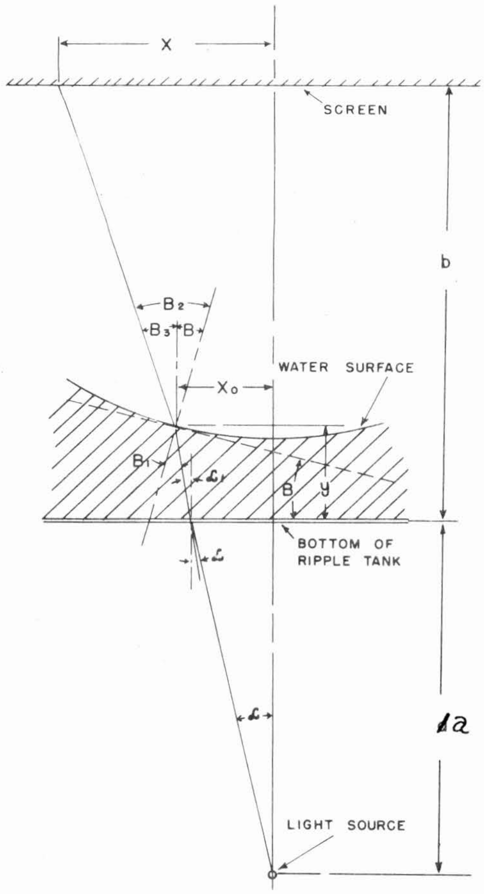


FIG. 199

hits the bottom of the tank at an angle α to the perpendicular. As the ray enters the water, it will be bent, due to the difference in refractive indices of the air and water. Its angle in the water is given by the equation

$$\frac{\sin\alpha}{\sin\alpha_1} = 1.33 \qquad (17)$$

in which 1.33 is the refractive index of water, compared to that of air.

Assume that the ray leaves the water in a region where the water surface is inclined at an angle β to the horizontal glass bottom. The angle β_1 between the ray in the water and the perpendicular to the water surface is

$$\beta_1 = 90^\circ - (90^\circ - \beta - \alpha_1)$$
$$= \alpha_1 + \beta \qquad (18)$$

the angle β_2 at which the ray leaves the water surface is again given by the refractive index, i.e.,

$$\frac{\sin\beta_2}{\sin\beta_1} = 1.33 \qquad (19)$$

and its angle β_3 with the vertical optical axis is

$$\beta_3 = \beta_2 - \beta \qquad (20)$$

Since all angles are small, it can be assumed that the sines, and tangents are equal to the corresponding angles. Furthermore, the glass bottom and the depth of water in the basin can be neglected, as their effects are small compared to the distances a and b between the light, the basin, and the screen.

The distance x from the optical axis at which the ray hits the screen is

$$x = x_0 + b \tan\beta_3 \qquad (21)$$

If the relationships of equations (17) to (20), together with the above trigonometric approximations, are substituted in (21) this becomes

$$
\begin{aligned}
x &\cong x_o + b\left[\beta_2 - \beta\right] \\
&\cong x_o + b\left\{1.33\,(\alpha_1 + \beta) - \beta\right\} \\
&\cong x_o + b\left\{\alpha + 0.33\,\beta\right\} \qquad (22)
\end{aligned}
$$

The angles α and β can be approximated as follows.

$$\alpha \cong \tan\alpha = \frac{x_o}{a} \quad \text{and } \beta \cong \tan\beta = \frac{\partial y}{\partial x_o}$$

If these relationships are introduced into equation (22), it becomes

$$x \cong x_o\left(\frac{a+b}{a}\right) + 0.33b\,\frac{\partial y}{\partial x_o} \qquad (23)$$

Equation (23) can be used to calculate the paths of the light rays and, consequently, the light distribution on the screen for any arbitrary known configuration of the water surface in the ripple tank.

The most noticeable feature of the ripple tank photographs is the series of lines of high light intensity, since they form the essential part of the pattern. The explanation of their significance can be found by studying equation (23). Since the distance a is great compared with the size of the tank, it may be assumed that the intensity of the light entering the bottom of the tank is constant over its entire area. If the water surface is undisturbed and, therefore, plane, the illumination of the screen also will be uniform. If the water surface is disturbed, this will no longer be the case, due to the varying angle of refraction at the surface. The intensity of the illumination at a given point on the screen can be found by differentiating equation (23), with respect to x_o. Thus

$$\frac{\partial x}{\partial x_o} = \frac{a+b}{a} + 0.33b\,\frac{\partial^2 y}{\partial x^2} \qquad (24)$$

the term $\frac{\partial x}{\partial x_o}$ can be interpreted to mean the width of the element on the screen illuminated by the light passing through a unit element of water surface located at x_o. If the surface is undisturbed, the second term of the right hand side of equation (24) is zero. The width of the strip on the screen illuminated by a unit strip of water surface under this condition is simply the geometric ratio $\frac{a+b}{a}$ for any position on the screen, which indicates a condition of uniform illumination. The screen intensity will be the reciprocal of this ratio. Now if the water surface is curved, the second term will not be zero. If the curvature is concave, as viewed from the air above, this term will be positive, which means that the light from a unit strip of surface will be spread over a wider area than normal on the screen, and, consequently, that portion of the screen will appear darker than when the water surface is undisturbed. If the water surface is convex upward, then the second term will be negative. As this term increases negatively from 0 to $-\frac{a+b}{a}$, the width of the screen strip illuminated by

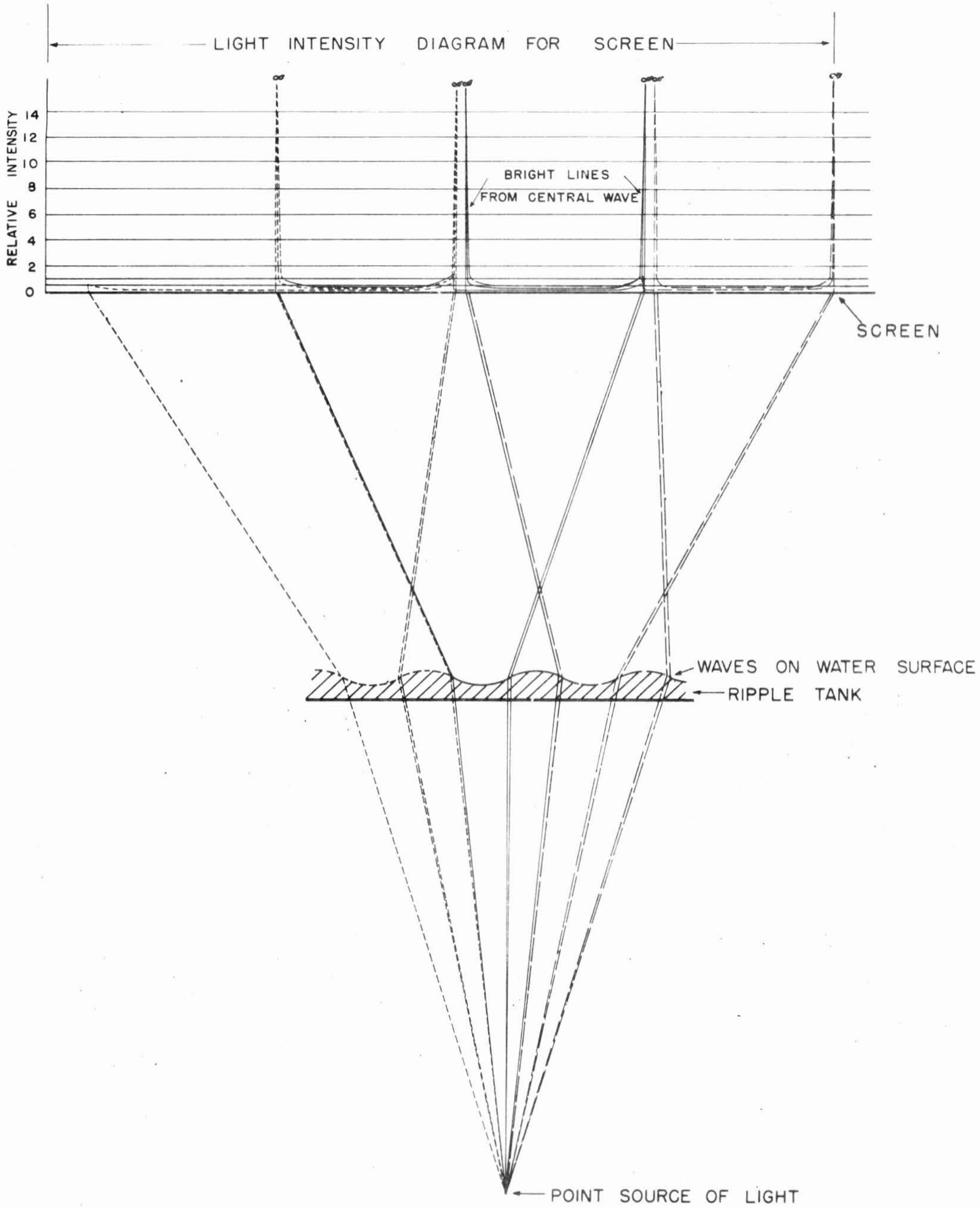


FIG. 200 TYPICAL RAY DIAGRAM FOR SIMPLE WAVE TRAIN SHOWING LIGHT PATTERN ON SCREEN

the unit water strip decreases from the normal value to zero, with a corresponding increase in illumination intensity to an apparent value of infinity at zero strip width. If this negative curvature is increased still farther, the screen strip widens again without limit. It will be seen that the reciprocal of equation (24) measures the intensity of the illumination on the screen.

To illustrate the significance of these relationships, consider a sine wave on the surface of the ripple tank of the form

$$y = A_o \sin\left(\frac{2\pi x_o}{L_o}\right) \tag{25}$$

In the equation A_o is the amplitude (1/2 wave height) and L_o the wave length. If this is differentiated and substituted in equations (23) and (24) they become

$$x = x_o\left(\frac{a+b}{a}\right) + 0.33bA_o \frac{2\pi}{L_o} \cos\left(\frac{2\pi x_o}{L_o}\right) \tag{23'}$$

$$\frac{\partial x}{\partial x_o} = \frac{a+b}{a} - 0.33bA_o \frac{4\pi^2}{L_o{}^2} \sin\left(\frac{2\pi x_o}{L_o}\right) \tag{24'}$$

In the ripple tank used in this study, dimension a is about 200 inches and b about 150 inches. If a wave having a length L_o of 2 inches and an amplitude A_o of .015" is assumed, these became

$$x = 1.75\ x_o + \frac{3\pi}{4} \cos(\pi x_o) \tag{23''}$$

$$\frac{\partial x}{\partial x_o} = 1.75 - 7.40 \sin(\pi x_o) \tag{24''}$$

Figure 200 shows a ray diagram for a train of these waves, as calculated from these values. The rays passing through the central wave are shown by solid lines, those passing through the right hand wave by dashed lines, and those through the left hand wave by dotted lines. The distances a and b have been shortened greatly in respect to the wave length for the sake of compactness. The wave amplitude is considerably increased to make it visible. The diagram above the screen shows the intensity of illumination as given by the reciprocal of equation (24"). Note that each wave produces two high intensity bands on the screen, and that these bands do not necessarily fall inside the area normally illuminated by the light passing through the undisturbed water surface covering the same as the wave. It is also evident that the distance between the two bright bands is a measure of the relative amplitude of the wave with respect to the wave length. For each wave length, there will be a minimum height of wave which will produce a single sharp bright line. Lower waves will produce only faint patterns of alternating light and dark bands, with one of each per wave. Higher waves will all produce the two lines per wave as shown in the diagram. For still higher waves, the pattern becomes very difficult to interpret, because of the overlapping of the lines for adjacent waves.

All of these variations can be observed in Figures 30 to 38 inclusive, beginning at Page 52.

APPENDIX II - MODEL BASIN AND INSTRUMENTAL EQUIPMENT

A. DESCRIPTION OF MODEL BASIN

The three sets of large models used in this study were all constructed in an outdoor model basin which is part of the Hydraulic Structures Laboratory of California Institute of Technology. A plan of this basin is shown in Figure 201. The main part of the basin is 40 ft. square. An extension 14 ft. wide and 22.5 ft. long on one side provides room for constructing a model river or channel in case one discharges along the shoreline within the model area. Water inlets and outlets are provided at the head of this river extension, along the entire opposite, or seaward end, of the basin and along a portion of one of the adjacent sides. These facilities make it possible to introduce river flows, simulate tidal prisms in bays or estuaries in cases where such bays would extend beyond the normal confines of the model, and to produce the ebb and flow of the tidal cycle in the entire basin. An 18-inch waterproof wall is provided around the area. A carriage which spans the entire basin runs on rails on top of the wall. This carriage serves both as a working platform with which to reach any point in the basin and also as a support for an observation and photographic boom. Figures 202 and 203 show a general view of the basin and the carriage with the boom. Water is supplied to any one of the three openings through individual sets of a large and a small Venturi meter. These meters are in parallel and have separate control valves so that the amount of flow can be maintained accurately at any desired rate. Small rates of flow can be withdrawn from the "ocean" outlet through another Venturi meter. This is useful for the purpose of withdrawing an amount equal to the continuous inflow of a river in order to maintain the constant water level in the basin. In this particular model, most of these facilities were unused or served merely as service connections to fill and empty the basin.

B. WAVE MACHINE

In order to produce the wave trains to cover the range of frequency desired for the present study, a wave machine was constructed. The basic design was similar to the wave machine that had been constructed for some previous studies. However, the old machine proved to be unsuitable for the present work because it had been designed to cover a much lower range of frequencies and thus was not rigid enough to withstand the vibration resulting from the higher speeds of operation needed for this work. The new machine was built around a steel truss that spanned the full distance of the basin and rested on the carriage rails. A vertical plunger was used to produce the wave. The oscillating system itself was designed to be complete without the plunger in place so as to make it possible to change plunger shapes, plunger length

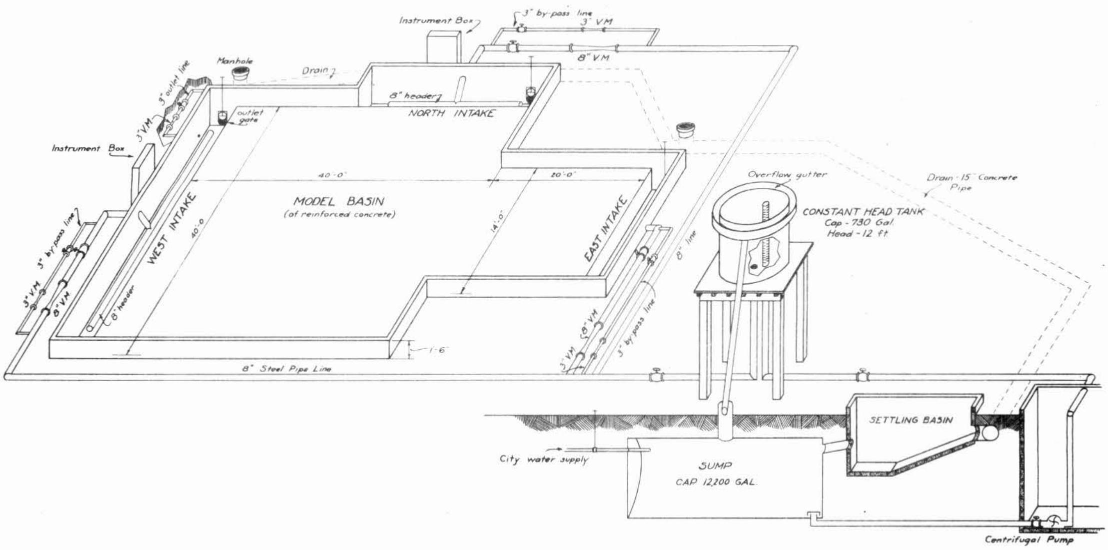


FIG. 201 DIAGRAM OF MODEL BASIN

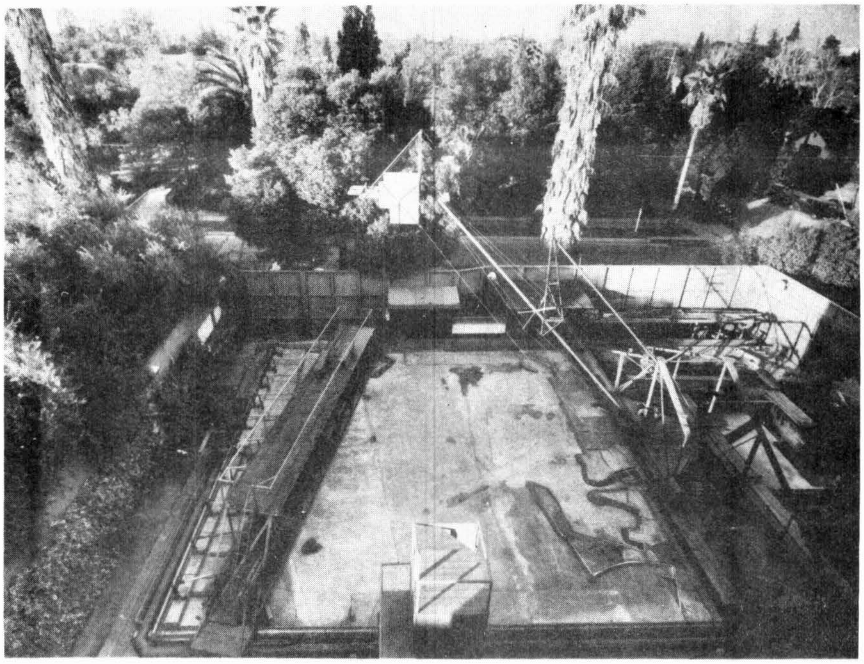


FIG. 202 GENERAL VIEW OF MODEL BASIN

FIG. 203 GENERAL VIEW OF OBSERVATION CARRIER TOGETHER WITH BOOM

and make other modifications without difficulty. An electric drive was provided through a $1\frac{1}{2}$ horsepower motor with a variable speed V-belt transmission and an integral gear reduction. This system had an output speed covering a range from 112 r.p.m. to 450 r.p.m. The motor and transmission was connected through a flexible coupling to a four-speed selective gear box, each step of which had the ratio of 3.5 to 1. Thus, the output shaft and the gear box could be operated any desired speed from 2.61 to 450 r.p.m. A disc was fastened to this output shaft which carried the crank arm which oscillated the main shaft of the wave machine. This connection was not direct, but through a variable stroke device which permitted a continuous variation in amplitude from 0 to 1.5 inches measured at the plunger. Simple grease-sealed ball bearings were used on all the connecting rods from the motor to the main shaft and from the shaft to the plunger elements in order to avoid play and roughness of operation. For the same reason, elastic radius arms were used to restrict the horizontal motion of the plunger. Figure 204 is a general view of the wave machine, Figure 205 a close-up of the motor transmission and variable stroke device, and Figure 206 shows the details of the main shaft and plunger connecting rod and radius rods. Various plunger shapes were constructed in an effort to produce wave trains with clean and smooth profiles, especially in the higher frequencies. The final shape adopted was a simple triangular cross-section with an adjustable front face which can be set at any angle from about 5 degrees to forty-five degrees to the vertical. The rear face was vertical and the apex down. In all cases a back

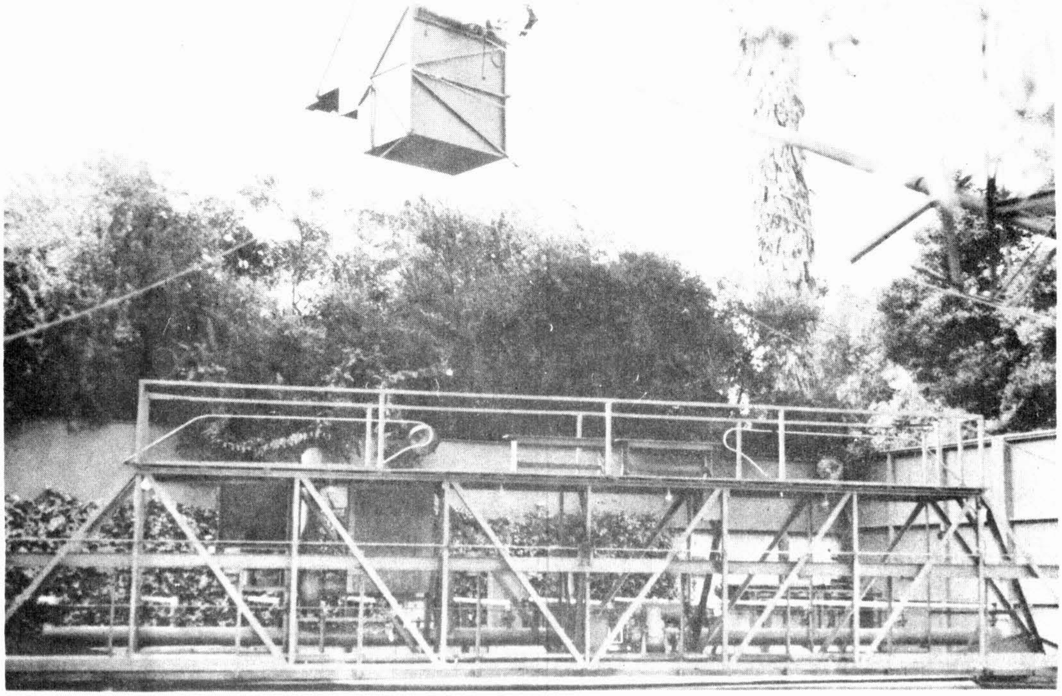


FIG. 204 GENERAL VIEW OF WAVE MACHINE

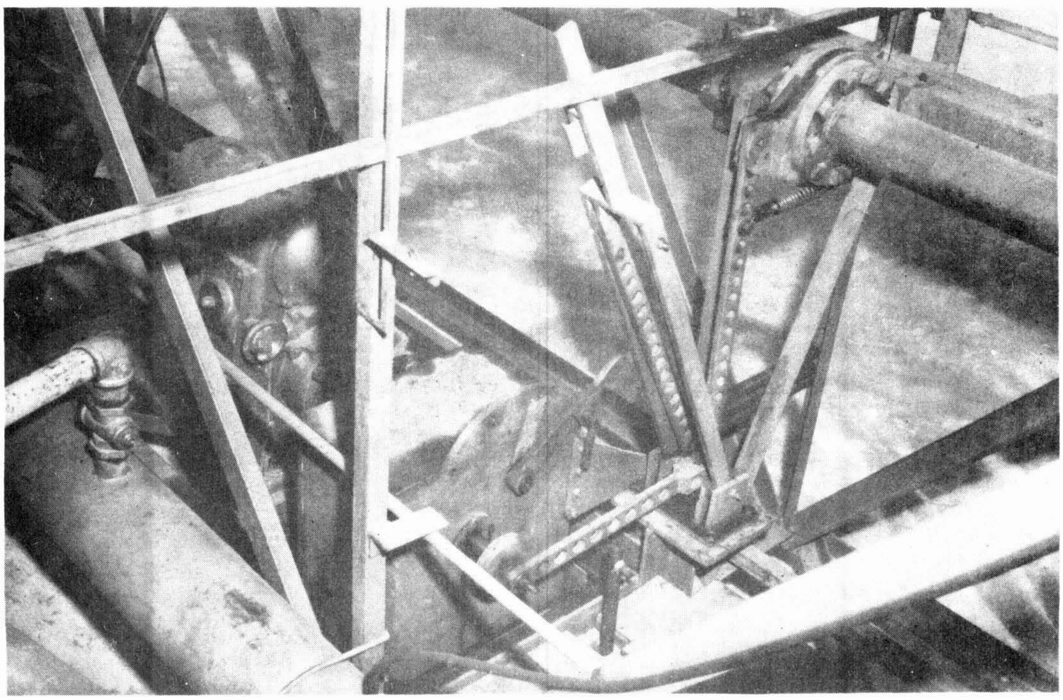


FIG. 205 CLOSE-UP OF MOTOR TRANSMISSION AND VARIABLE STROKE DEVICE OF WAVE MACHINE

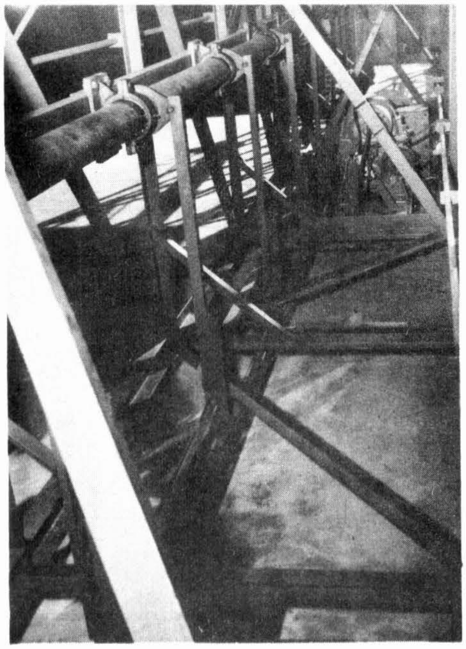


FIG. 206 CLOSE-UP OF MAIN SHAFT, PLUNGER CONNECTING ROD AND RADIUS RODS OF WAVE MACHINE

board was provided with only a small clearance between it and the rear vertical face of the plunger. This was for the purpose of confining the disturbance to a single wave per stroke emanating from the front face only.

C. SURGE MACHINE

Although the wave machine described in Section B of this appendix has a wide speed range, it is not sufficient to cover the requirements for the long-period surge waves. Furthermore, the longer the period of the wave, the more water has to be moved per wave. Hence, it was decided to construct a separate machine to produce the surges. This was developed using a completely different principle of action. Figure 207 shows a diagrammatic sketch of this device. It consists of an air-tight tank with a long, narrow opening on one side and a connection for an air pipe on the top. A series of these tanks is installed upon the ocean side of the model with the openings facing toward the model. The device is actuated through the air connection which goes from the tank to a control valve. One side of this control valve is open to the atmosphere. The other side leads to the suction connection of a centrifugal exhauster fan, which is capable of producing a suction pressure of about 8 inches of water. The control valve is operated by a surge cam which is cut to any desired profile required to produce the shape of surge indicated for the model study. As the surge cam revolves, the surge tank is connected alternately and gradually to the fan or exhauster and to the atmosphere. When the surge tank is connected to the exhauster, water enters the

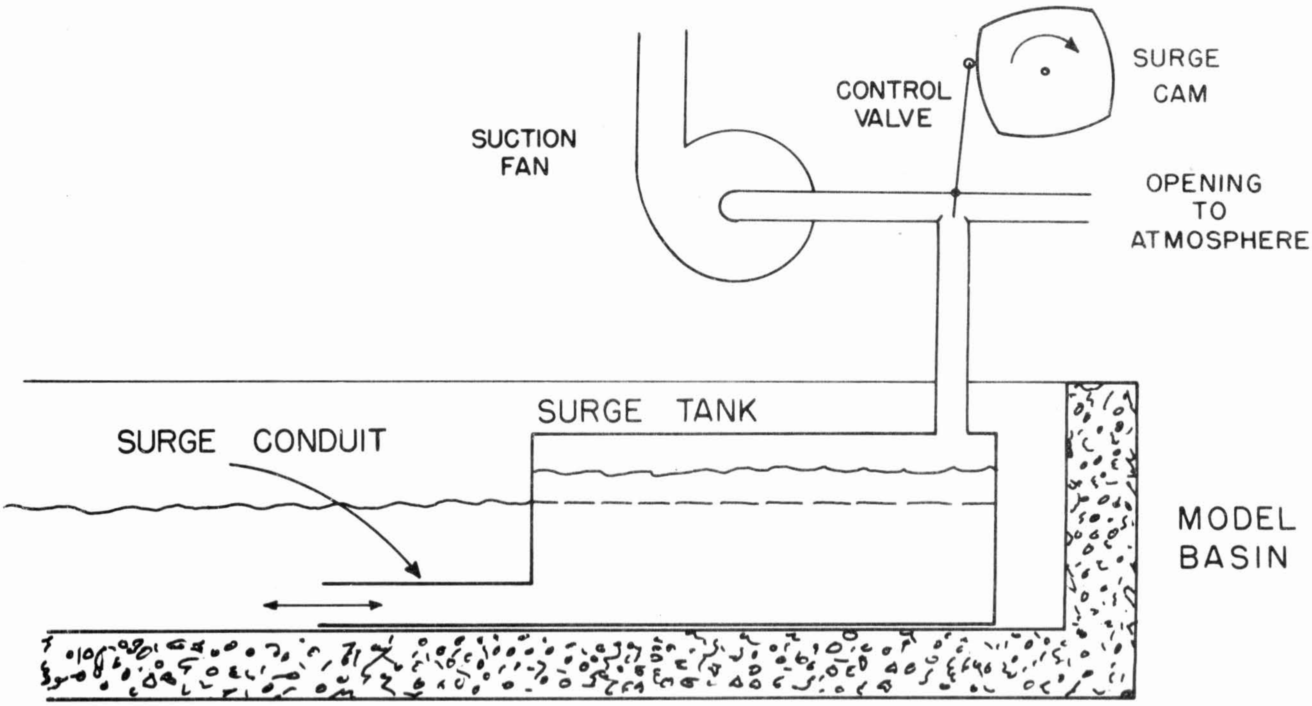


FIG.
DIAGRAM OF SURGE MACHINE

FIG. 207 DIAGRAM OF SURGE MACHINE

surge tank from the model basin through the surge conduit and as the control valve gradually shifts over to the atmospheric side air is readmitted to the surge tank, thus allowing the water to flow back to the model basin. By varying the speed and thus the intake pressure of the suction fan, and by changing the shape of the surge cam, any desired amplitude and wave profile can be produced. Furthermore, by varying the speed of rotation of the surge cam, the period of the surge can be controlled. When the desired combination has been secured for a given surge, the speeds of the fan and the cam both remain constant throughout the experiment. Figure 208 shows a series of these surge tanks installed in the model basin. The surge conduits may be seen running along the bottom of the basin. They are normally screened from view by a continuation of the sheet of metal seen at the left hand side of the picture. This inclined sheet, which projects through the water surface, is used as a wave damper to eliminate any extraneous wave motion produced by the plunger of the wave machine which is installed in front of the surge tanks. Figure 209 is another view of the surge tanks showing the connections between them and the manifold that leads to the control valve. The discharge pipe of the exhauster fan is also visible. Figure 210 shows the control valve, together with the surge cam with its variable speed motor and four-speed gear reducer. Figure 211 shows

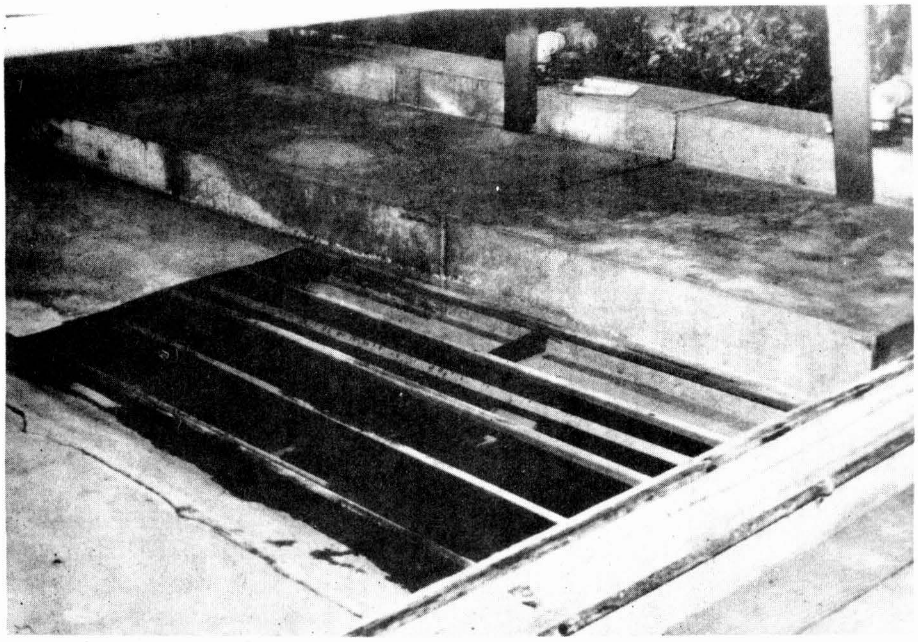


FIG. 208 SURGE TANKS

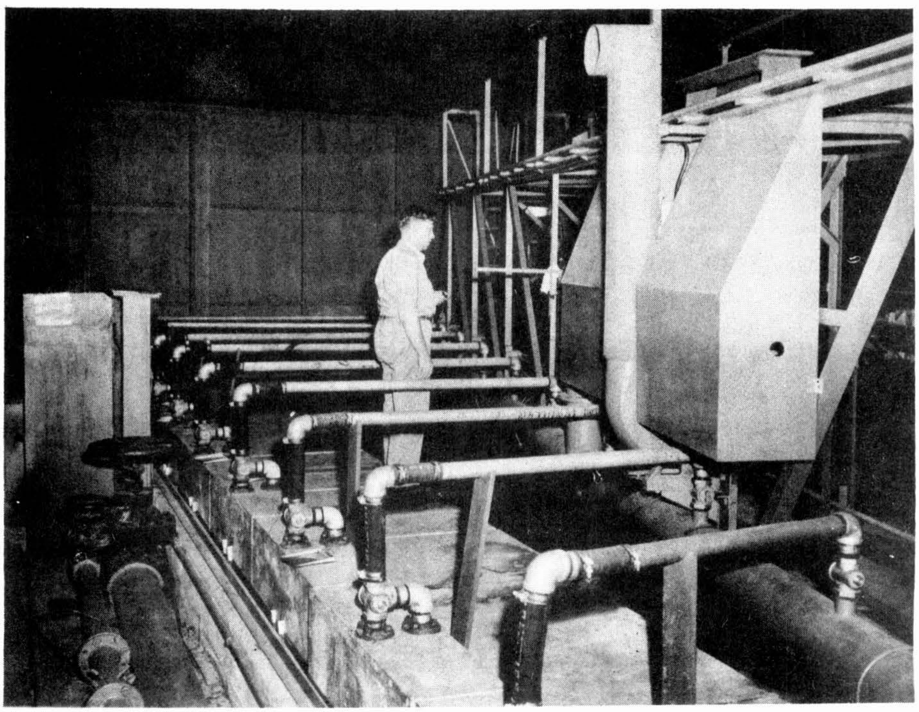


FIG. 209 SURGE TANKS

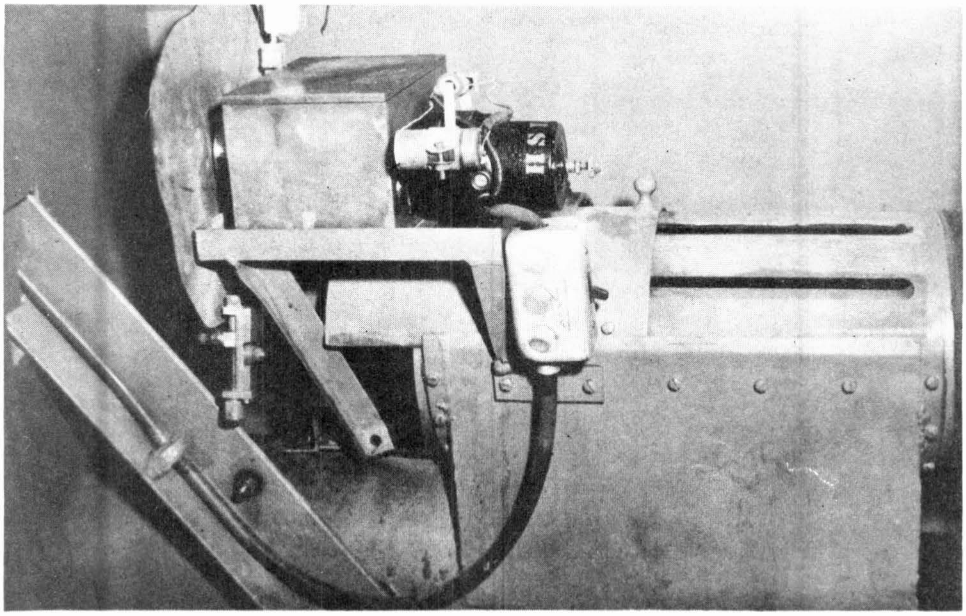


FIG. 210 SURGE CONTROL VALVE, CAM, VARIABLE SPEED MOTOR AND GEAR BOX

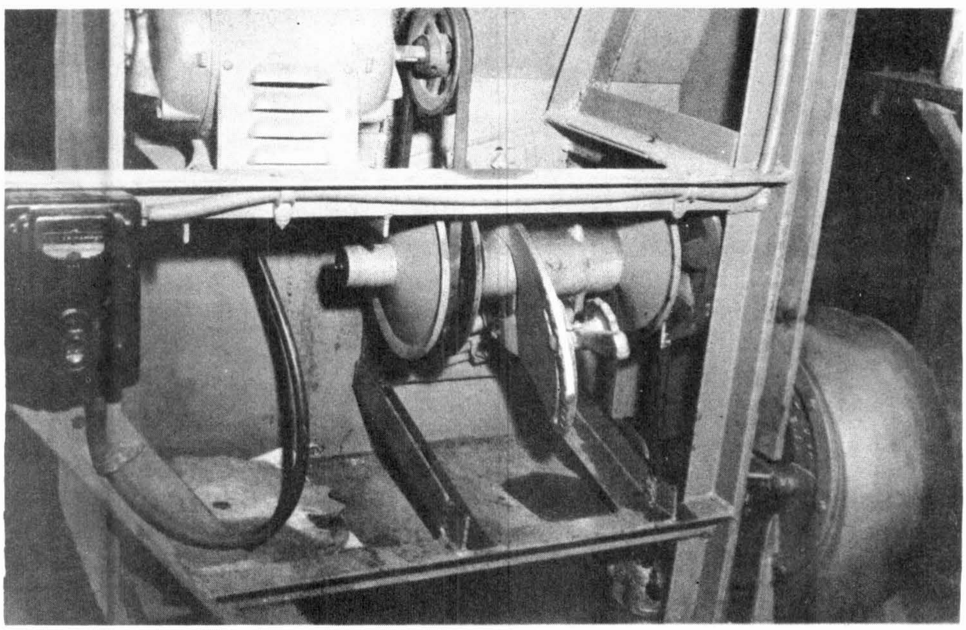


FIG. 211 EXHAUSTER FAN AND VARIABLE SPEED DRIVE

the exhauster fan with its open type variable speed V-belt drive.

D. EQUIPMENT FOR MEASURING VERTICAL WATER MOTIONS

The equipment for measuring the vertical water motion produced by the waves and surges was described very briefly on pages 78 to 80. The following is an amplification of this description.

1. GENERAL PRINCIPLES OF METHOD OF MEASUREMENT

This method consists essentially in measuring the amount of electrical current flowing between two parallel wires projecting vertically through the water surface. This construction will be called a "conductivity element". Within limits, if a constant voltage is impressed across such an element, the amount of current that will flow will be proportional to the length of wire immersed in the water and hence, if the wires are stationary, to the change in elevation of the water surface. In order to approach as close as possible to two dimensional conditions, the immersed ends of the wires are insulated.

2. CONSTRUCTION OF CONDUCTIVITY ELEMENTS

Figure 52, Page 80, shows the details of construction of two types of these elements. The element marked 34 is a normal wave-measuring element. The exposed wires are of platinum-ruthenium alloy, sealed into a glass tube at the upper end. A light glass bridge sealed to the lower ends serves both as an insulator and as a spacer to maintain the wires parallel. The spacing is 1/4 inch center to center and the exposed length of the wires is 1 inch. The platinum wires are soldered to a two-conductor flexible cord inside of the glass tube and the tube is filled with insulating pitch. Element T-5a, shown in the same photograph, is designed to be used totally immersed, and serves as a calibrating element to indicate changes in water temperature or other factors that modify the basic conductivity of the water. Variations in the readings of this type of element are used to correct the readings of the wave measuring elements. Figure 212 shows the battery of these elements that was used in the study. Figure 213 shows a short element holder with five elements mounted on it. In the background will be seen a tripod for single element installation. To insure consistency in the readings, the elements were cleaned at frequent intervals by immersing them in a dilute solution of nitric acid. The platinum and glass construction made this method of cleaning very simple since these materials are not attacked by the cleaning solution.

3. ELECTRICAL CIRCUITS

The electric circuit required for the use of these measuring elements is extremely simple. It is shown in elementary form in Figure 214. It will be noticed that alternating current is used. This is to prevent the decomposition of the water due to the passage of the current. If decomposition were to occur, bubbles would

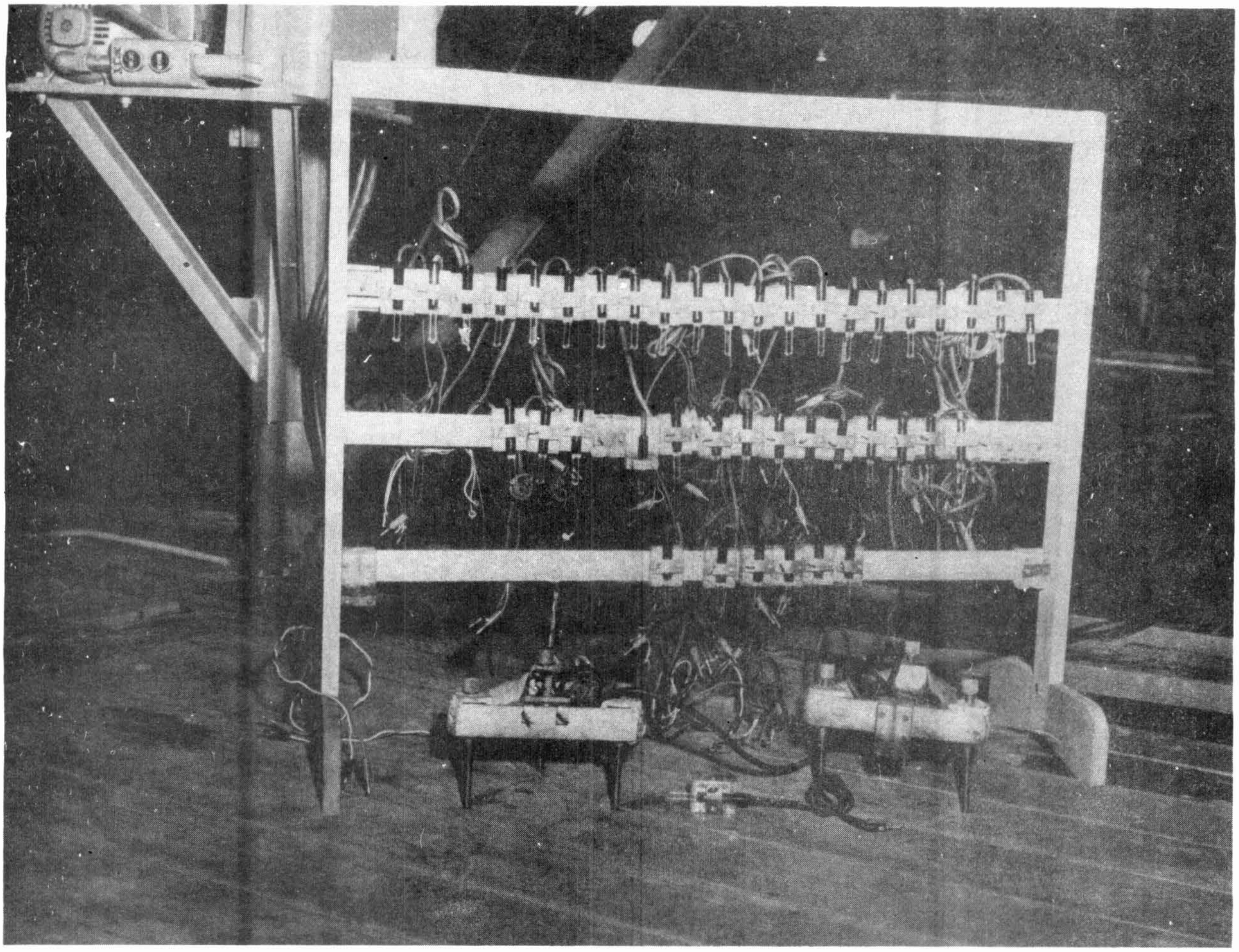


FIG. 212 CONDUCTIVITY ELEMENTS

collect on the wires, thus changing the area in contact with the water and, hence, the calibration of the elements. For a similar reason, i.e., to prevent heating and the formation of small bubbles of water vapor, the current density is kept as low as possible by using the minimum voltage that will give a reading on the galvanometer large enough to obtain the required accuracy.

Figure 215 shows a more complete circuit for a single element. The five-ampere Variac transformer was introduced into this circuit to provide voltage control for the entire system of elements. The circuit for each oscillograph galvanometer was individually controlled by a one-ampere Variac, so that the galvanometer readings could all be brought to the same calibration, thus eliminating considerable calculation. A transformer was introduced between the wave measuring element and the galvanometer to reduce the voltage across the wave elements and at the same time to increase the current through the galvanometer so as to secure the desired sensitivity.

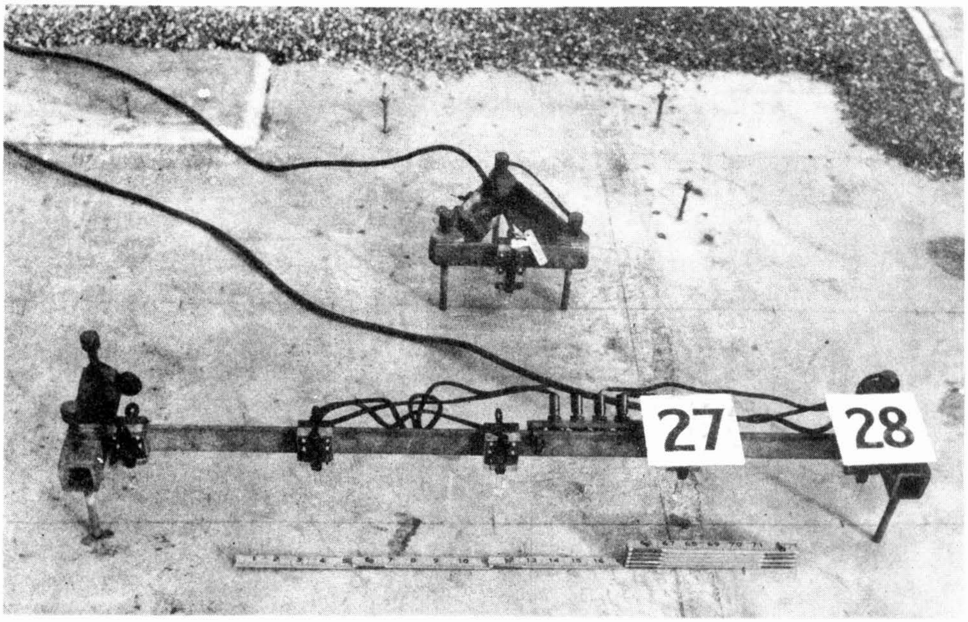


FIG. 213 ELEMENT HOLDERS

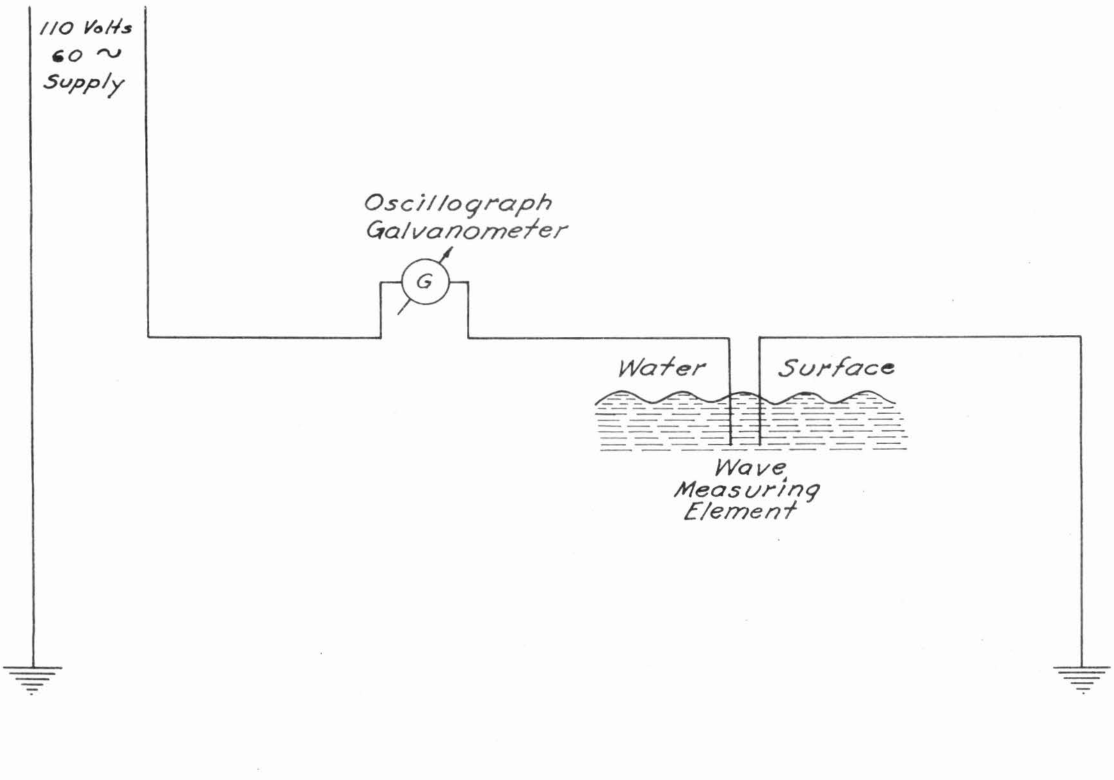


FIG. 214 ELEMENTARY MEASURING CIRCUIT

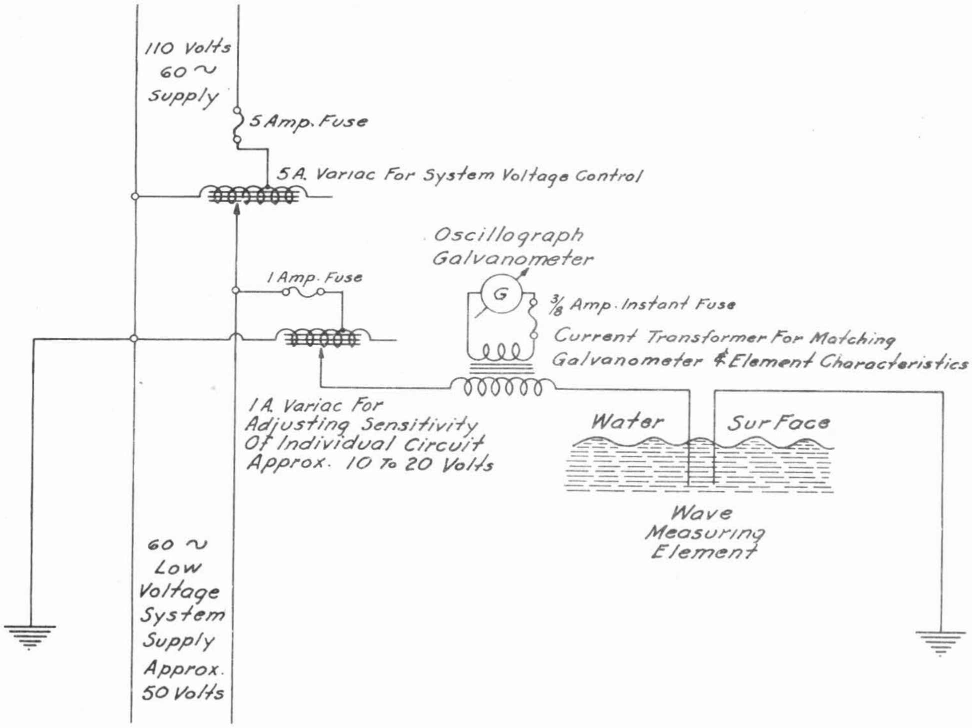


FIG. 215 COMPLETE CIRCUIT FOR SINGLE ELEMENT

Figure 216 shows the final circuit adopted for the system. A four-element oscillograph was used. For each galvanometer circuit, twelve wave measuring elements were brought to a 12-pole selector switch. Thus, forty-eight elements could be installed in the basin at one time and records obtained in groups of four elements at a time. Figure 217 shows a series of sample records obtained with this equipment. It will be observed that each record is an envelope of the 60-cycle oscillations of the galvanometer mirror. In other words, the 60-cycle is the carrier frequency upon which the wave frequency is superimposed. In the initial stages of the development of this equipment, a 2000-cycle carrier frequency was used. This proved to offer no advantages and had the disadvantage of poor regulation and low current capacity. Hence, it was discarded in favor of the 60-cycle line frequency.

It will be seen that each record carries timing lines along one edge. The small divisions represent an interval of .1 of a second. Different recording speeds were used for different wave periods.

E. PHOTOGRAPHIC EQUIPMENT

Two types of special photographs were made during the course of this study: (1) wave pattern records, (2) horizontal water motion measurements. Both of these series of photographs were taken from the photographic boom shown in Figure 203.

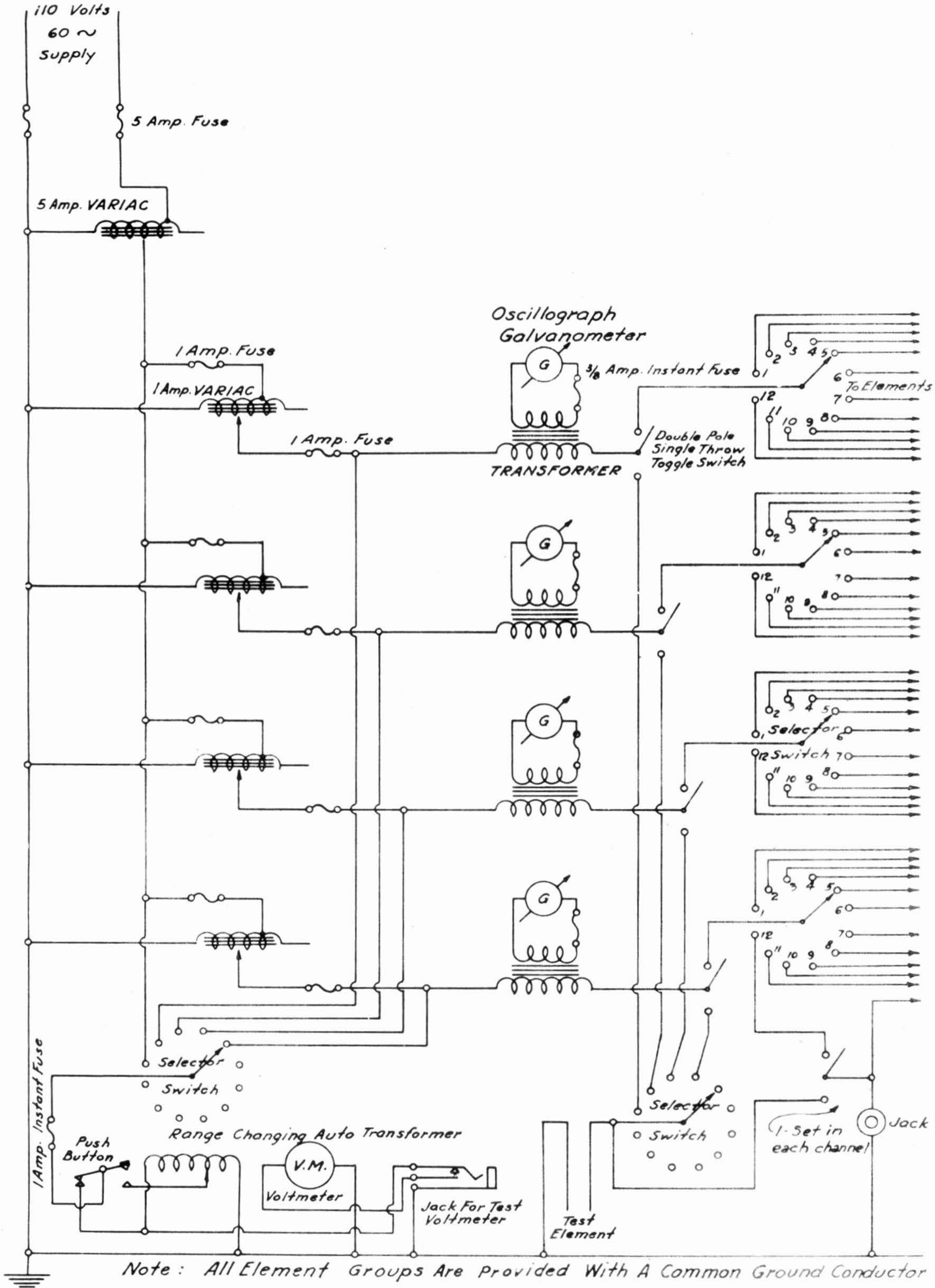
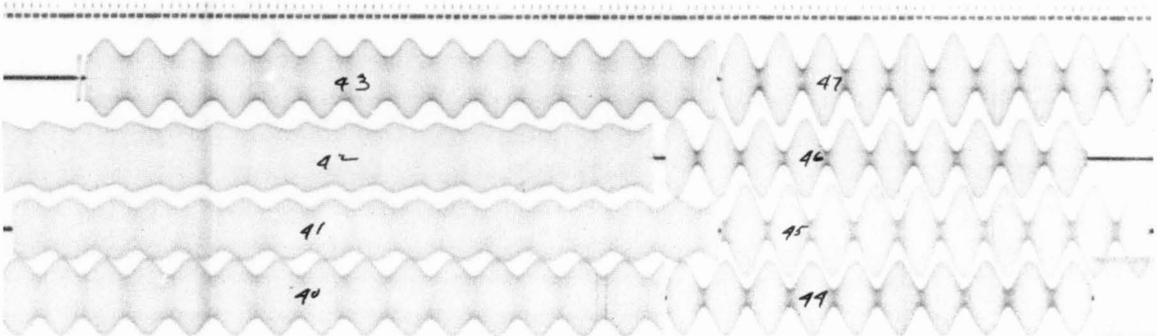
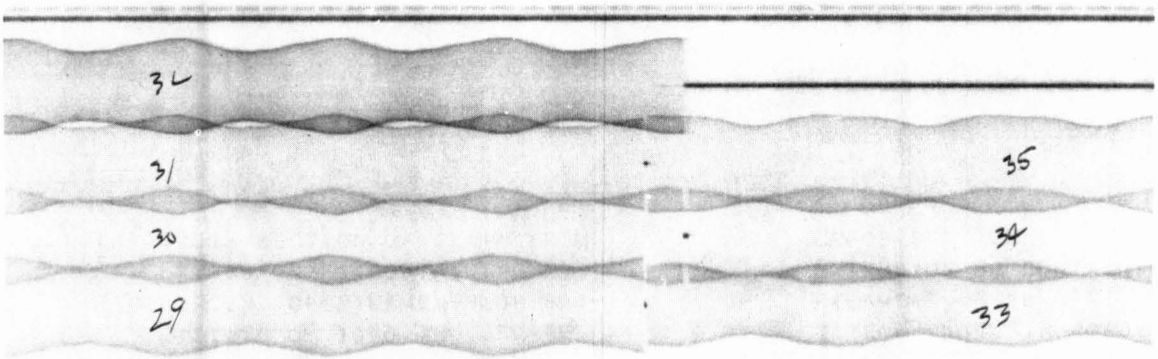


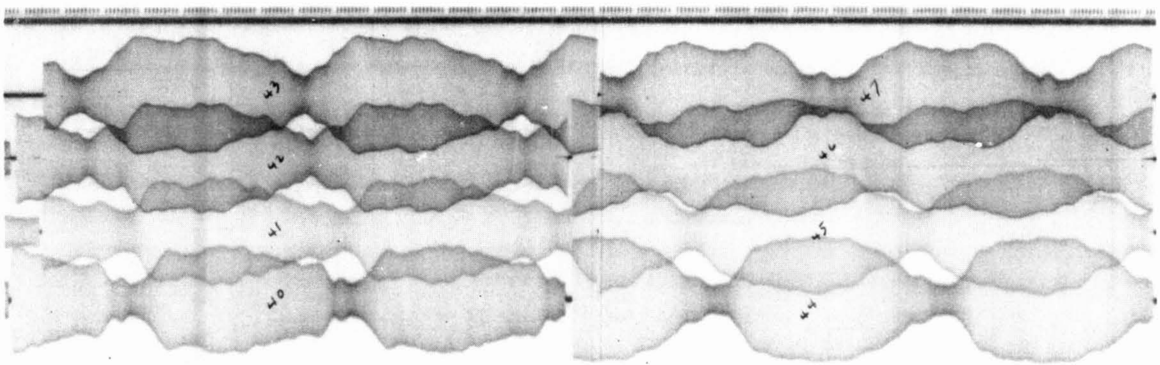
FIG. 216 FINAL ADOPTED CIRCUIT



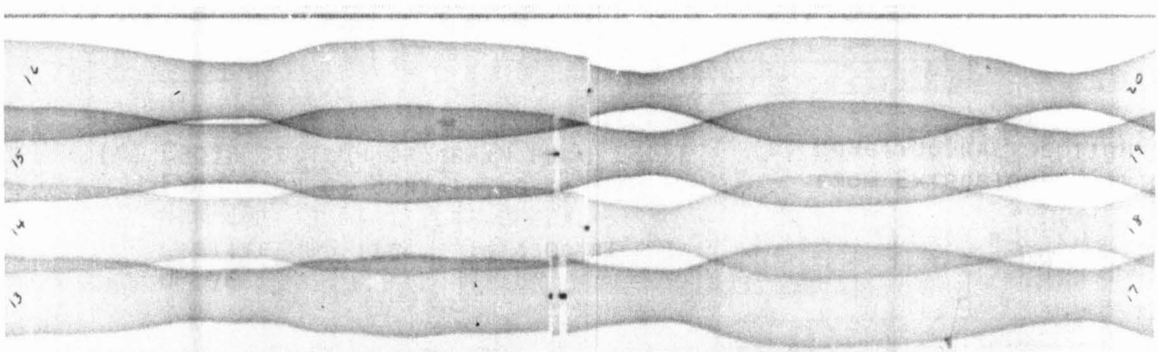
15 SECOND WAVES



3 MINUTE SURGES



3 MINUTE SURGES AND 15 SECOND WAVES



6 MINUTE SURGES

FIG. 217 SAMPLE OSCILLOGRAPH RECORDS

1. WAVE PATTERN PHOTOGRAPHS

All of the wave-pattern photographs were taken with an Army Air Force Aircraft camera, Model K-17b, which takes a 7" x9" picture. This was equipped with a 6 inch focus Metrogon lens. This wide angle lens made it possible, within the limits of the permissible elevation of the boom, to cover the entire model area with a single photograph. By adjusting the height of the boom to a pre-determined value above the water level, the exact scale was established in which 1 inch on the photograph equals 5 ft. in the model. Figure 218 shows the photographic cage on the end of the boom as seen from below and Figure 219 the camera installation on the cage. It will be observed that the aircraft camera is provided with a vacuum back which insures a flat negative.

To obtain the contrasting lights and shadows required to bring out the details of the wave pattern, these photographs were all taken at night with controlled illumination. Special sheetmetal reflectors were constructed for this purpose for use with banks of 500 watt, photoflood lights. Figure 220 shows these reflectors in place for a wave-pattern shot. They were movable and were adjusted to secure the best lighting conditions for each set of experiments. From five to eight of these banks were used simultaneously, which represents an input of about thirty kilowatts of illumination power.

2. HORIZONTAL WATER MOVEMENT MEASUREMENTS

The photographic technique used in measuring the horizontal water motions has been described briefly on pages 78 and 146 to 147. The equipment used for making the required photographs is also shown installed in the photographic cage in Figures 218 and 219. It consists of an 8"x10" view camera mounted on an extension to the aircraft camera table, and three reflector-type photoflood lamps mounted just below it. In general, only one of these lamps was required to make the exposure. The reflectors used are shown in Figure 221. They are mounted in small wood floats which have beveled edges to prevent float-to-float contact at the water surface. This bevelling eliminates the tendency of the surface tension of the water to hold groups of floats together. The reflectors themselves were small buttons of plastic commonly used in roadside traffic signs. Figure 222 shows a photograph on the model basin as the reflectors were being distributed over the basin surface in preparation for a record photograph.

F. CONSTRUCTION OF HARBOR MODEL

The same basic methods of construction were used for Models 1, 2 and 3 in the basin. In all cases, the shoreline was made of wooden sections installed on the concrete floor of the basin and fitted together in accordance with surveyed control points. The bottom was then brought to proper contours, according to the best

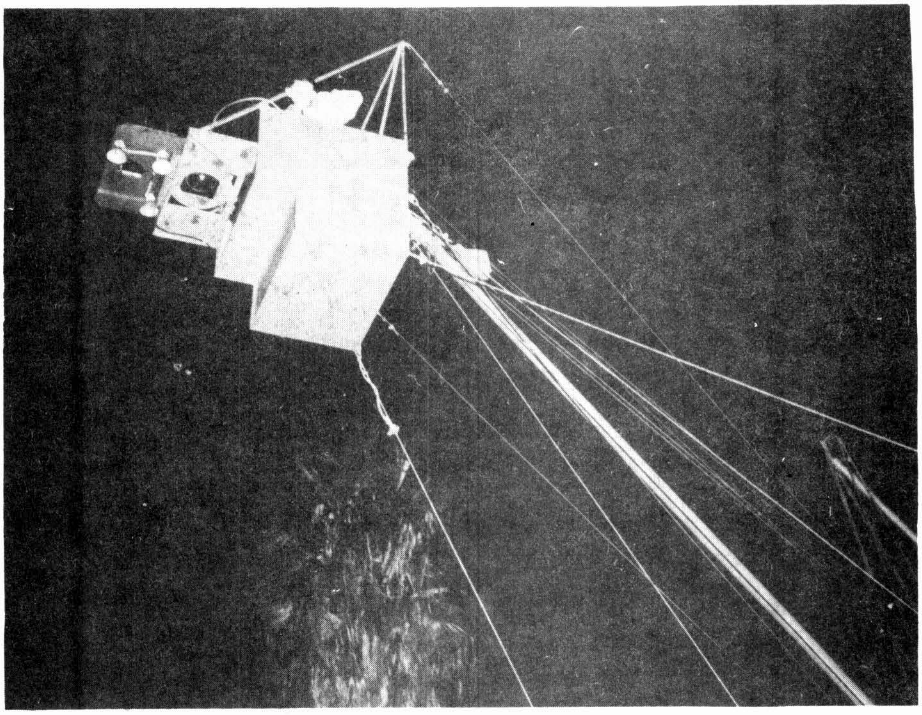


FIG. 218 PHOTOGRAPHIC CAGE

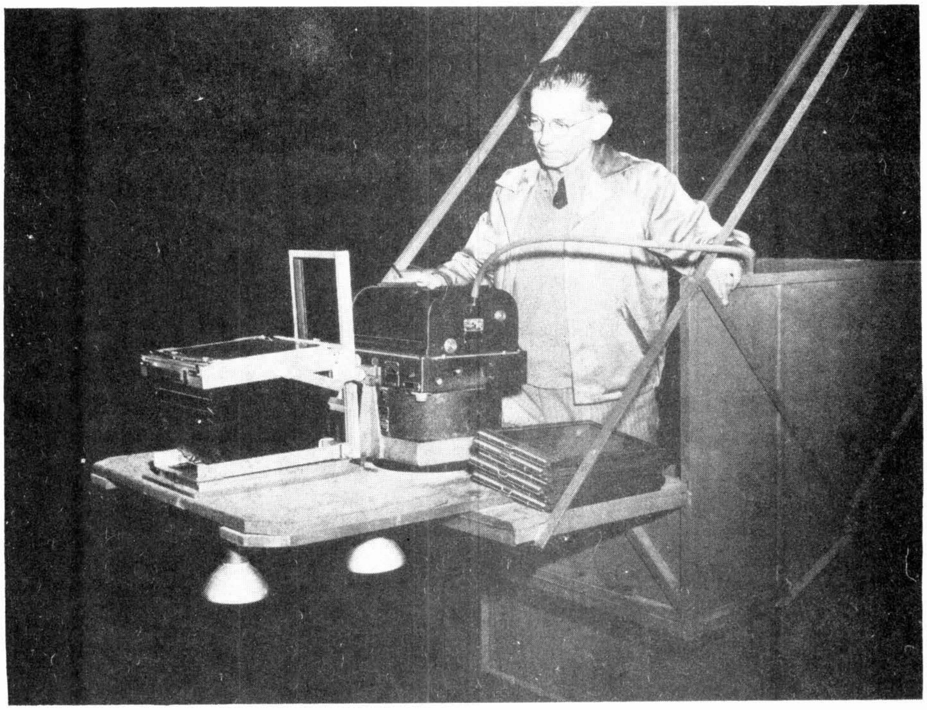


FIG. 219 CAMERA INSTALLATION

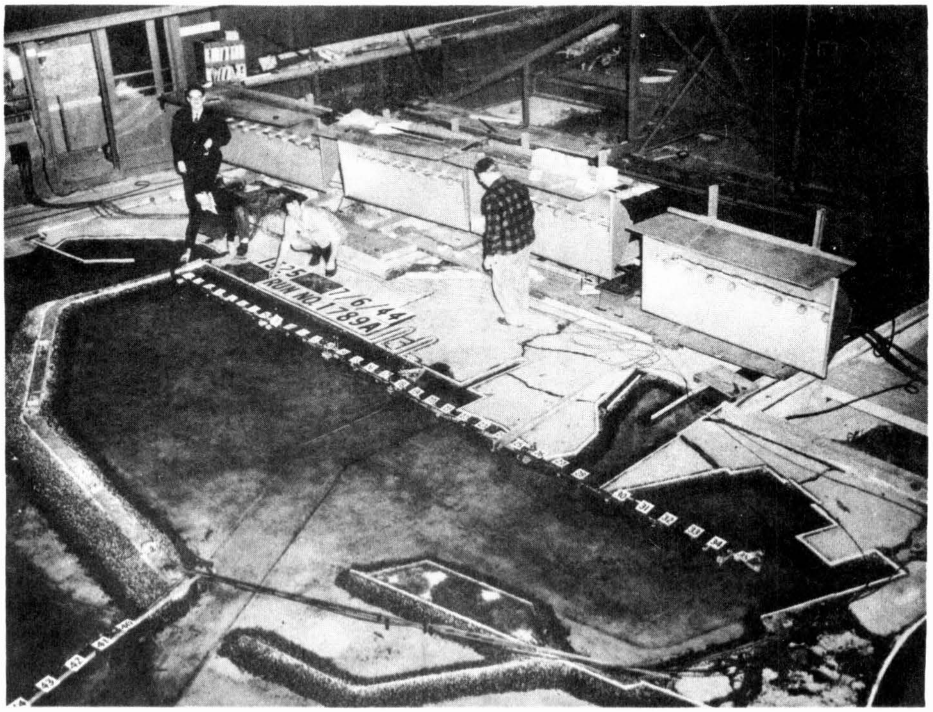


FIG. 220 FLOODLIGHT REFLECTOR INSTALLATION FOR WAVE-PATTERN PHOTOGRAPHS

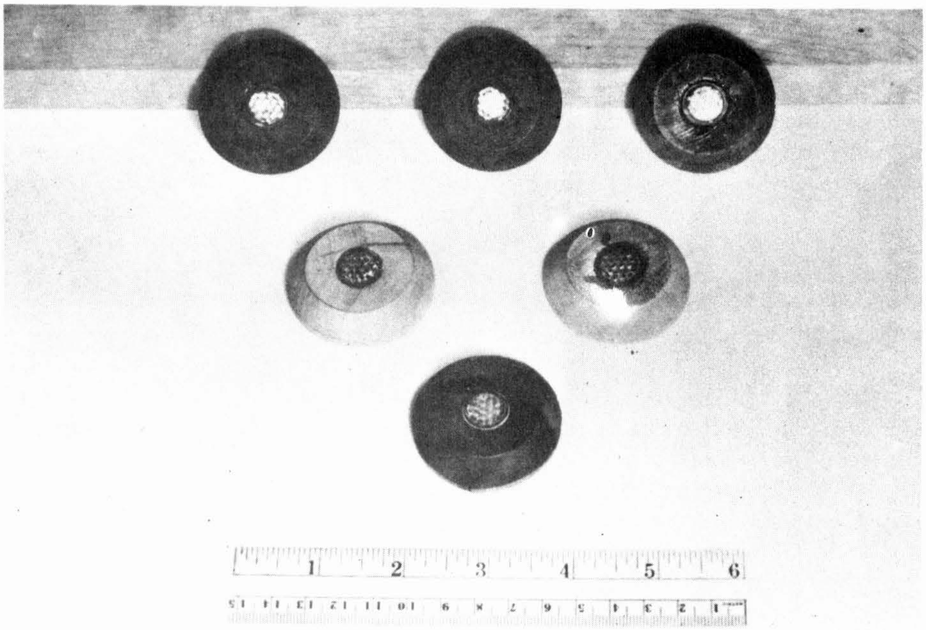


FIG. 221 REFLECTOR FLOATS

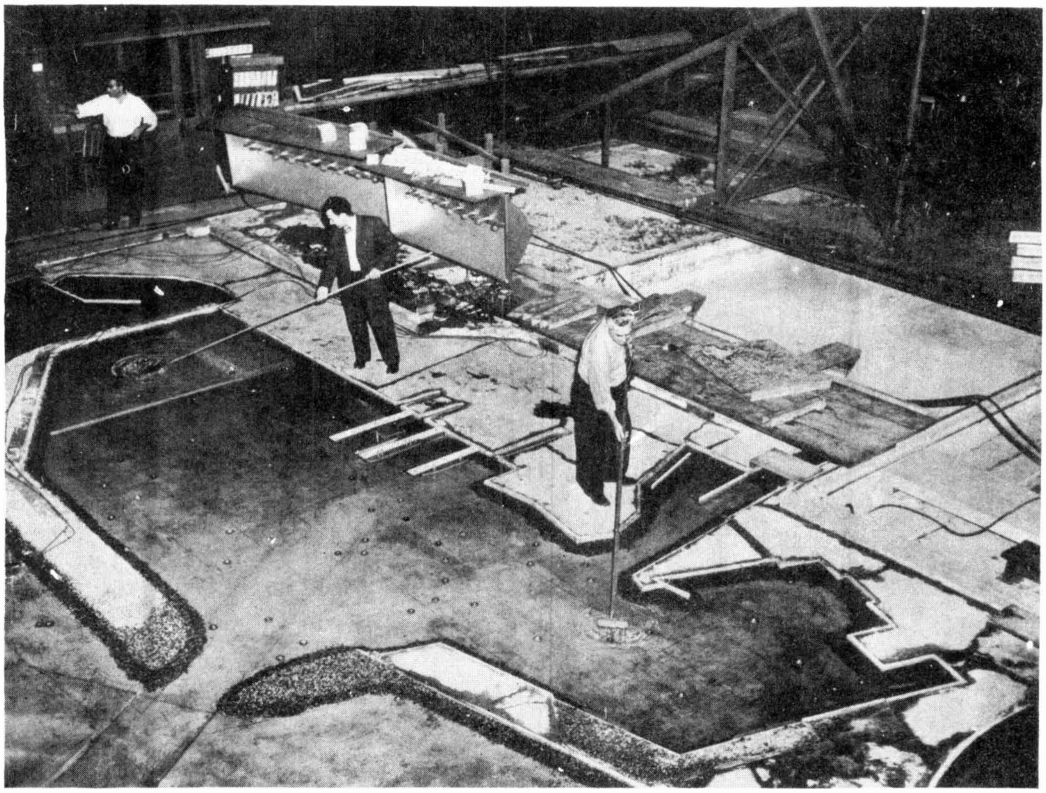


FIG. 222 REFLECTOR DISTRIBUTION

underwater surveys available, by the use of a sand fill. When the desired contours were obtained, they were stabilized by a cement coating which was made relatively thick in the working areas and thinner in the expanse of ocean toward the wave and surge machines. Figure 223 shows the erection of the wooden shoreline for Model 1. Figures 224 and 225 show the method of grouting some of the shoreline and breakwater sections to the basin floor. Figure 226 is the completed model and Figure 227 shows the details of the proposed basin to enclose the Naval Operating Base area. Figures 228, 229, and 230 show similar stages in the construction of Model 2.

G. MODEL SHIPS

A series of nine model ships was constructed for the study of ship motion in the harbor. As previously stated, they were made of wood and ballasted with lead shot until they had the correct waterline, metracentric height, and roll and pitch periods. The model ships were made in accordance with drawings furnished by the Naval Operating Base. This series of models is shown in Figure 231 and part of the fleet may be observed docked at Piers 1, 2 and 3 in Figure 232.

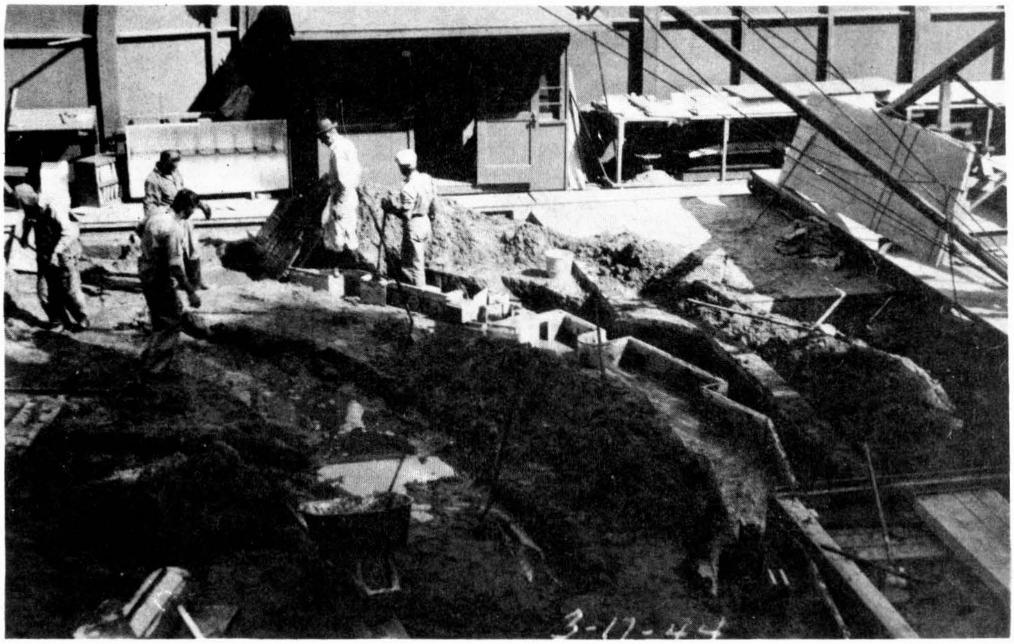


FIG. 223 MODEL NO. 1 SHORELINE CONSTRUCTION

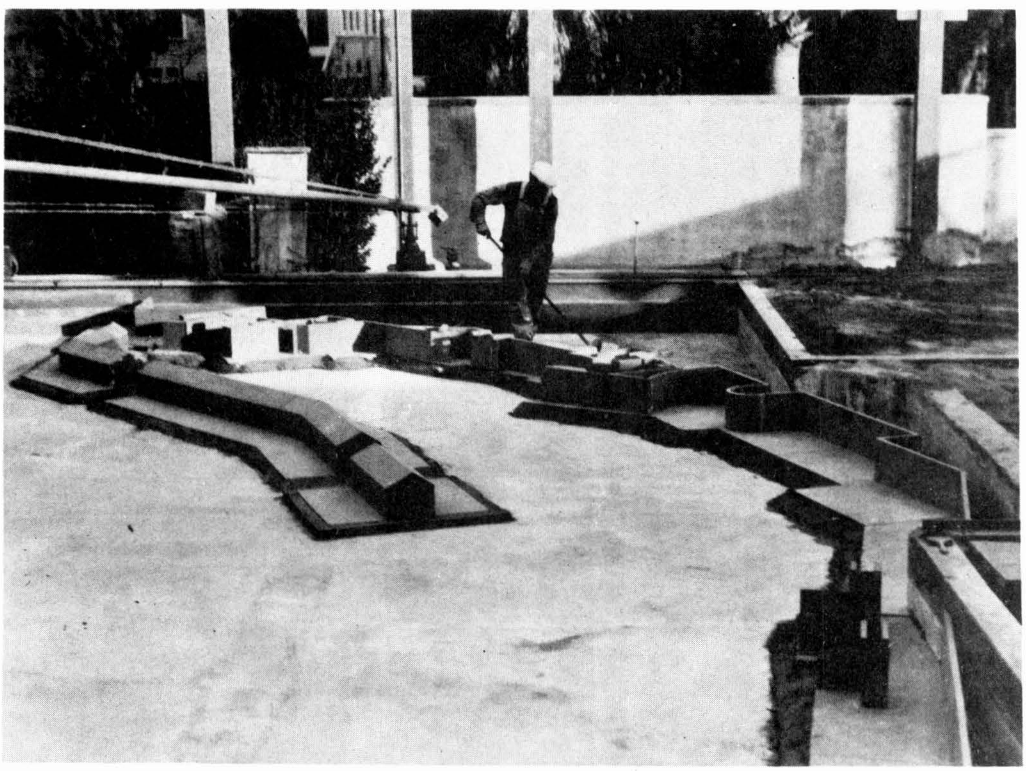


FIG. 224 METHOD OF GROUTING SHORELINE & BREAKWATER SECTIONS
MODEL NO. 1

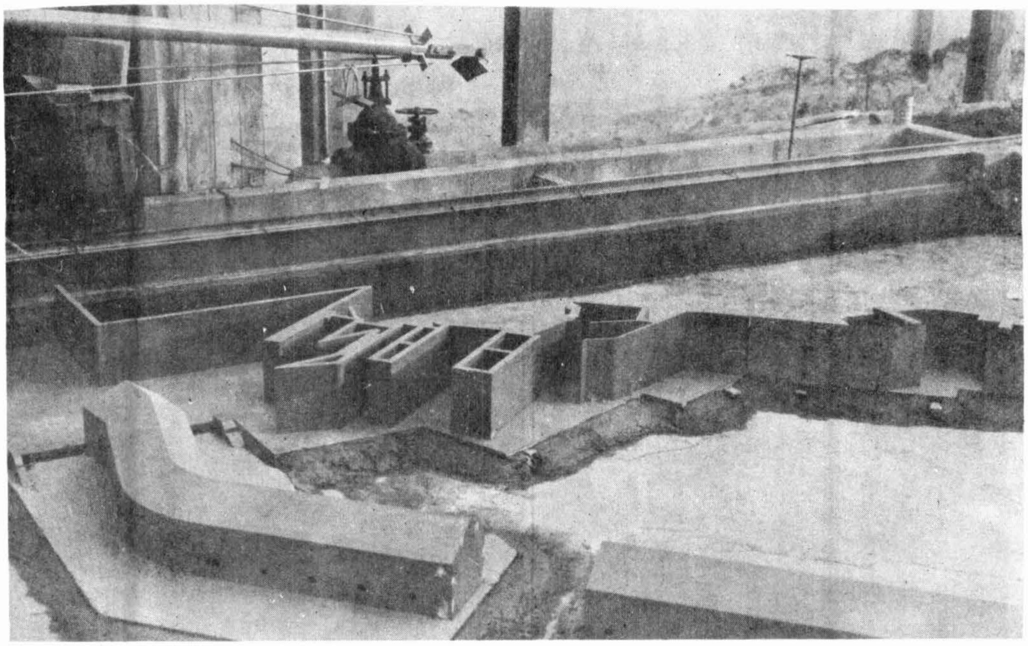


FIG. 225 METHOD OF GROUTING SHORELINE AND BREAKWATER SECTIONS
MODEL NO. 1

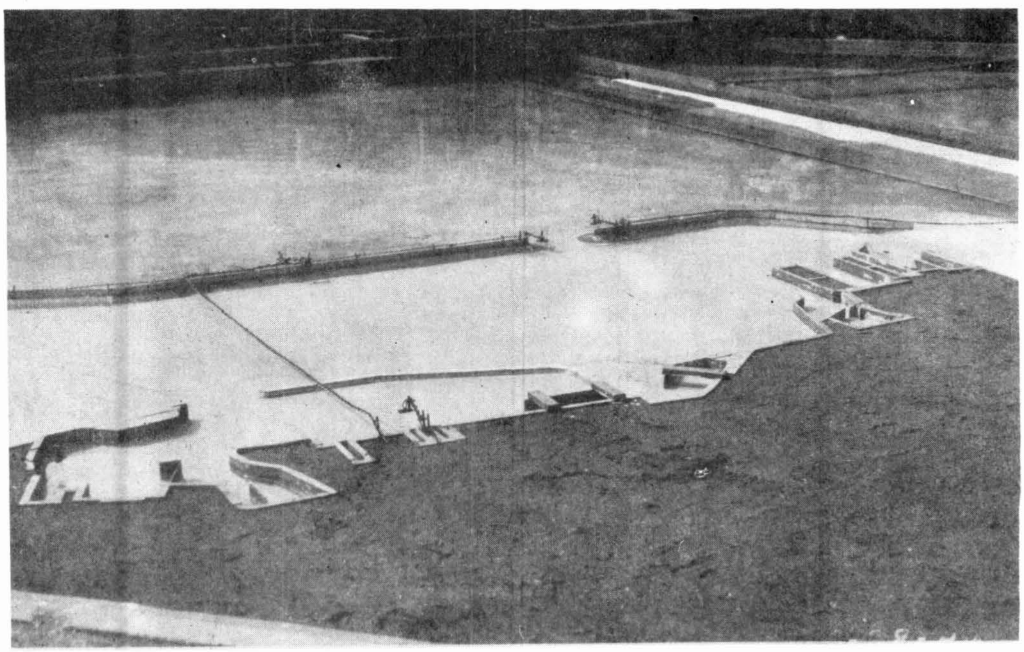


FIG. 226 MODEL NO. 1 SHORELINE

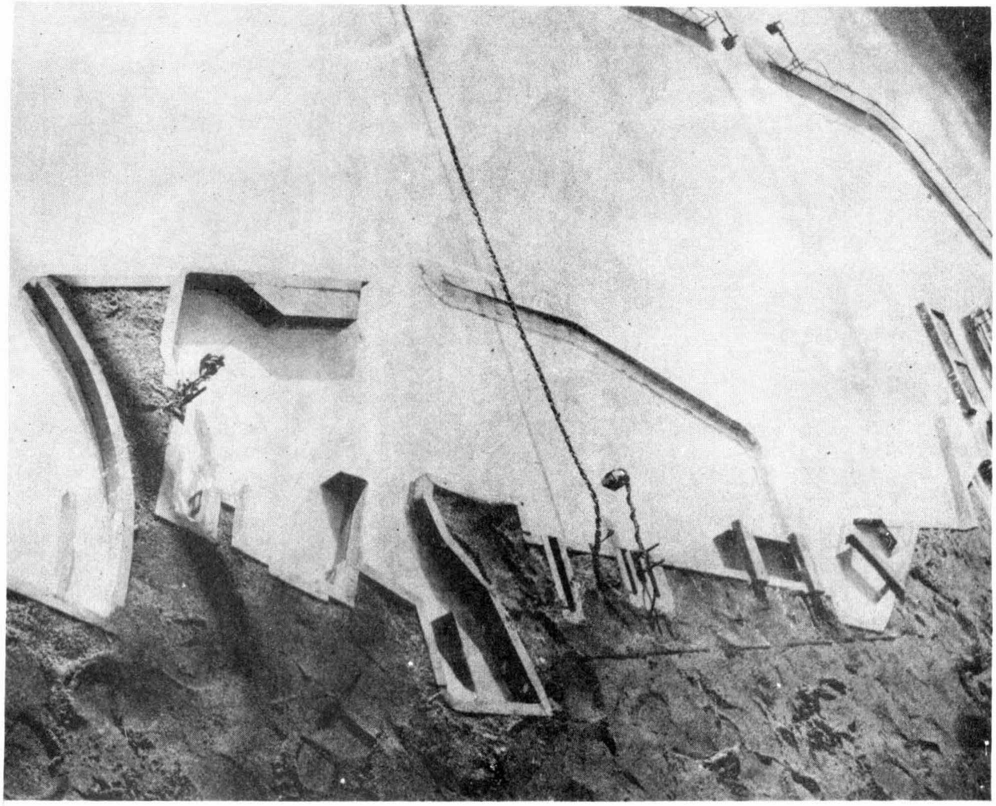


FIG. 227 MODEL NO. 1 - NAVAL OPERATING BASE AREA

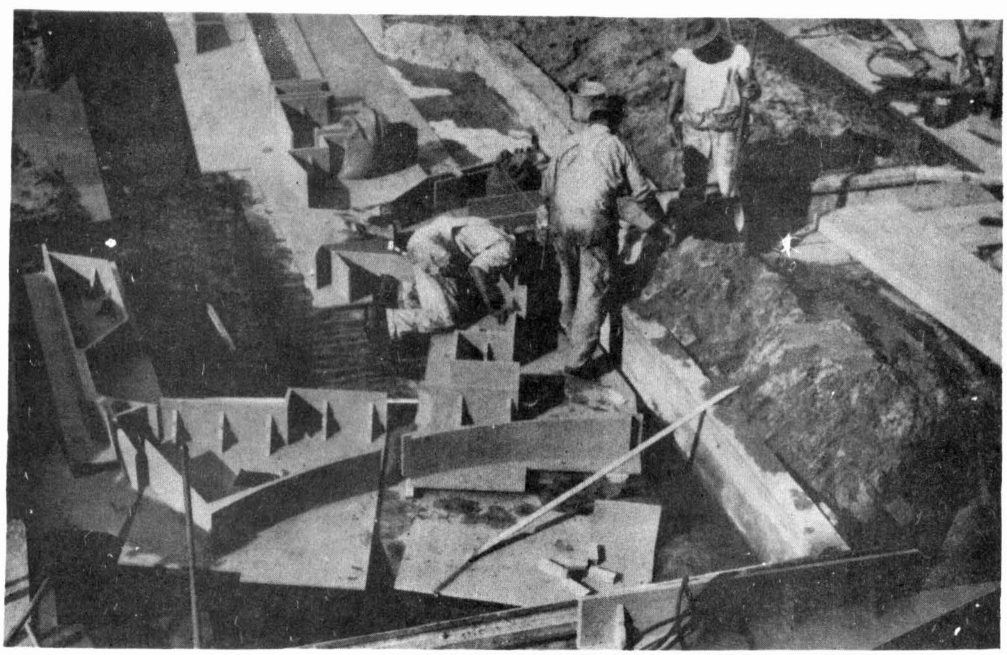


FIG. 228 MODEL NO. 2 - SHORELINE CONSTRUCTION

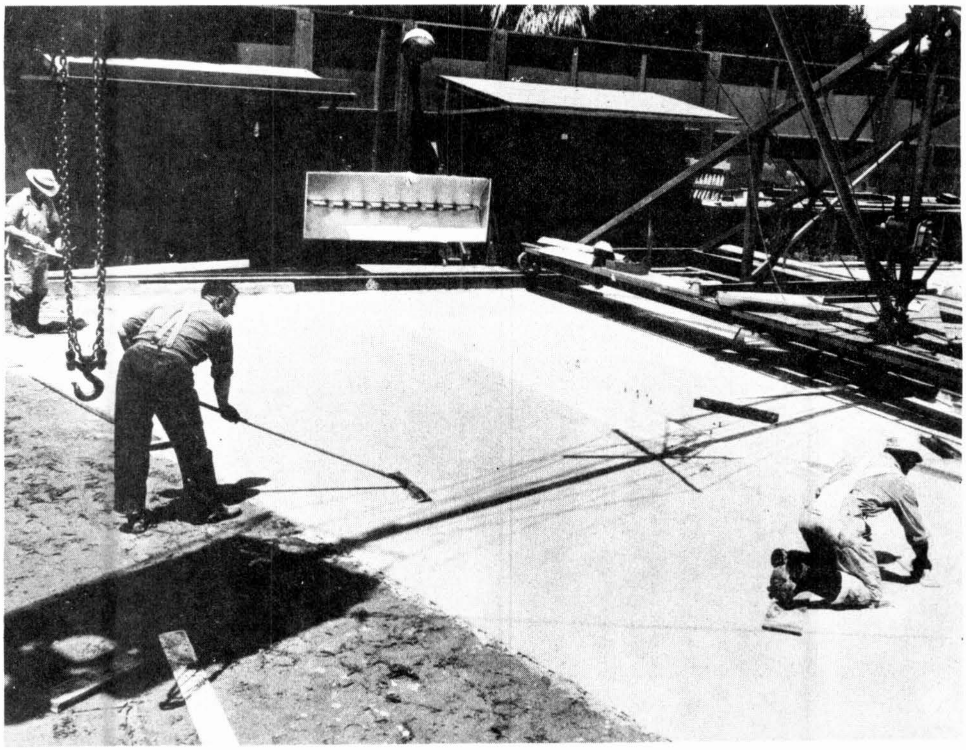


FIG. 229 MODEL NO. 2 - APPLYING CEMENT COATING

FIG. 230 MODEL NO. 2 - APPLYING CEMENT COATING

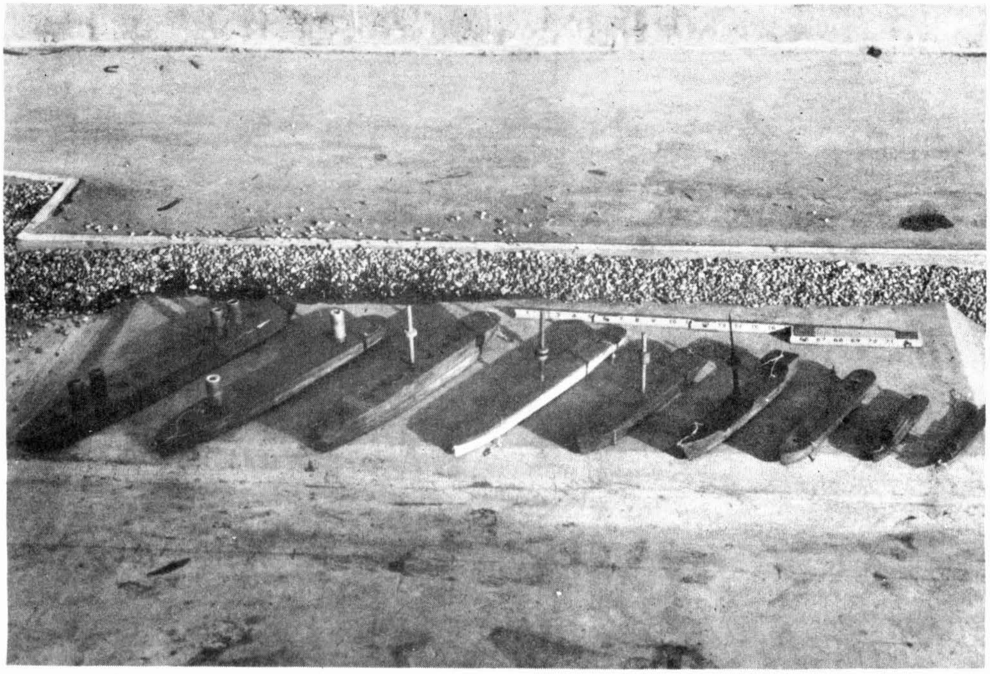


FIG. 231 SHIP MODELS

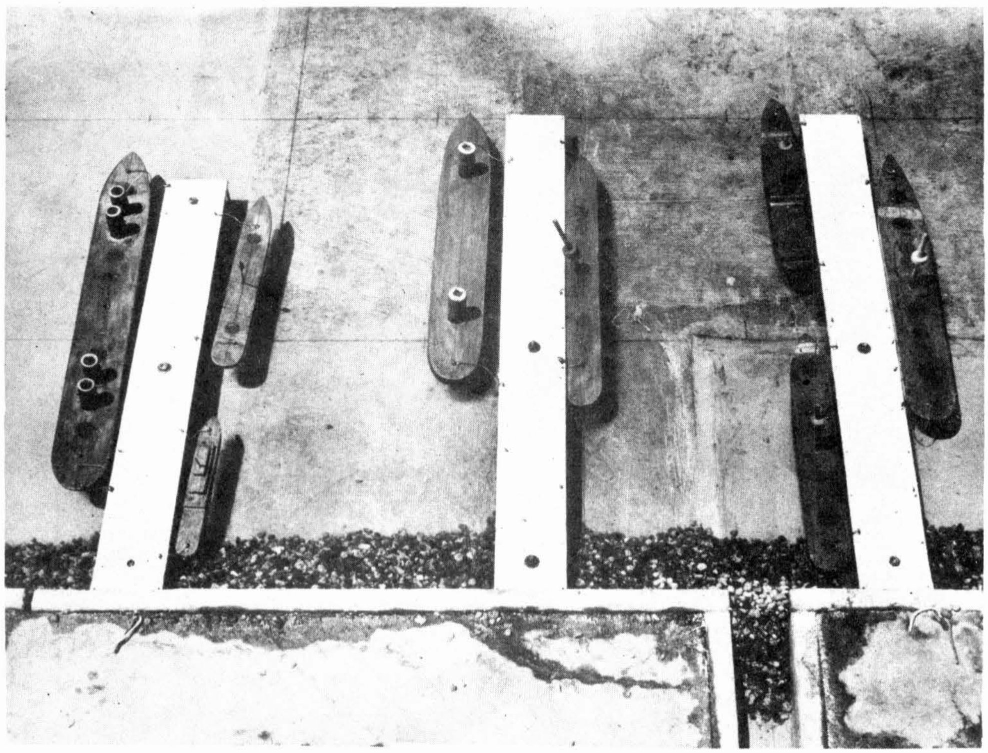


FIG. 232 SHIP MODELS MOORED AT PIERS 1, 2, AND 3

APPENDIX III -- TABLE OF RUNS -- MODELS 2 AND 3

For reference purposes, the accompanying chart has beenprepared to show the runs made on Models 2 and 3. The code accompanying the chart shows clearly the status of the runs, what data were taken, what computations were made, and what charts were prepared.

CORRELATION OF GROUP NUMBERS
OSCILLOGRAMS, REFLECTOR PHOTOS & CONTOUR MAPS

LEGEND

- X — VERTICAL AMPLITUDES - MODEL RUN SERIES (OSCILLOGRAM) AVAILABLE
- O — HORIZONTAL MOVEMENT - REFLECTOR PHOTOS AVAILABLE
- ⊗ — INDICATES BOTH X AND O AVAILABLE
- XMX — CONFIGURATION AND VERTICAL WATER MOVEMENT CHART PREPARED
- OMO — CONFIGURATION & HORIZONTAL WATER MOVEMENT CHART PREPARED
- ⊗MO — CONFIGURATION & HORIZONTAL WATER MOVEMENT CHART PREPARED ALSO VERTICAL MOVEMENT DATA AVAILABLE
- ⊗MX — CONFIGURATION AND VERTICAL WATER MOVEMENT CHART PREPARED ALSO VERTICAL MOVEMENT DATA AVAILABLE
- ⊗M⊗ — CONFIGURATION AND BOTH VERTICAL AND HORIZONTAL WATER MOVEMENT CHARTS PREPARED
- ----- — INDICATES COMPARISON CHART

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